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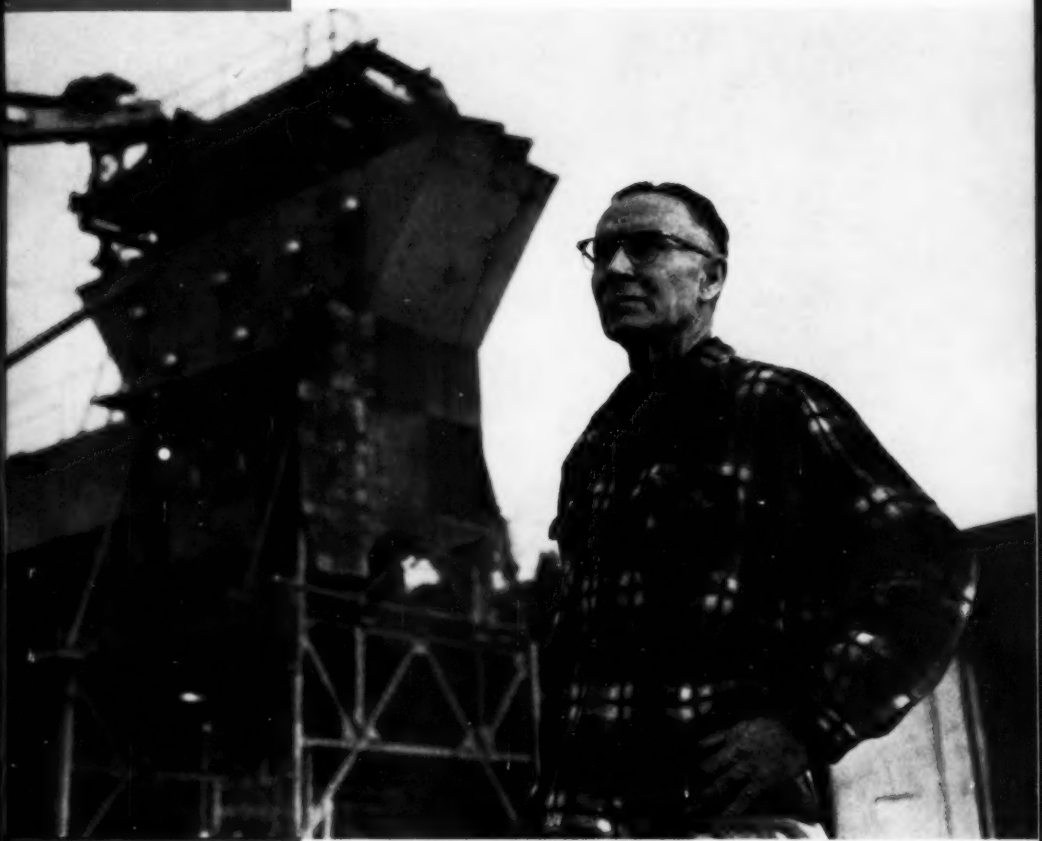

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Coming next month in the JOURNAL

Past president JOE W. KELLY looks forward as he discusses the uniqueness of concrete, pointing out that the best years of this widely used material are still ahead, in his outgoing presidential address, "Universal material—Cosmopolitan society."

L. A. ASHTON and S. C. C. BATE present results of an investigation of "The Fire Resistance of Prestressed Concrete Beams" with post-tensioned cables. Objective of the investigation was to obtain information on the design of beams for buildings of high fire risk requiring fire resistance of 4 hr for structural elements.

N. S. LONG and T. W. HOWELL present an on-the-job construction review of "Concrete for the Mammoth Pool Power Tunnel," an integral part of the Southern California Edison Co.'s Big Creek Hydroelectric Power Project which develops the hydroelectric potential of the upper San Joaquin River.

CHARLES WILSON gives a step-by-step description of the analysis and design of prestressed concrete bridge members as performed by a small scale electronic computer in his paper, "Prestressed Concrete Beams by Electronic Computer."

"The Response of Concrete Shear Keys to Dynamic Loading" by ROBERT J. HANSEN, EDWARD G. NAWY, and JAYANT M. SHAH reports on a series of tests on three types of concrete shear keys under static and dynamic loading comparable to that experienced in a building subject to air blast loading from nuclear weapon.

C. P. THORNE reports on extensive investigation of "Concrete Properties Relevant to Reactor Shield Behavior," commenting on the type of concrete best suited to the particular requirements of reactor shielding.

On the Cover—LEWIS H. TUTHILL, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento, who was installed as President of the American Concrete Institute for a 1-year term at the February, 1961, annual meeting.

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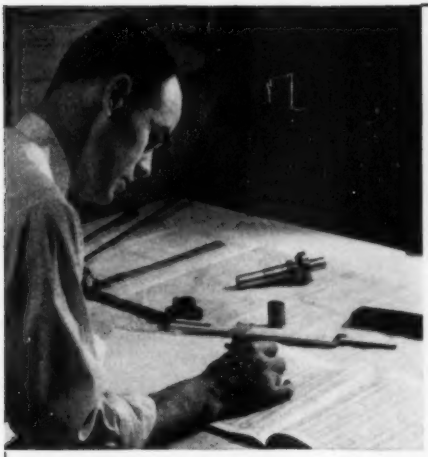

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Load Tests of Patterned Concrete Masonry Walls

By R. O. HEDSTROM

Laboratory tests were performed to determine compressive and flexural strengths of concrete block walls laid in nine different patterns. Flexural tests were made on wall panels subjected to bending across a vertical and a horizontal span. Two types of mortar were used. Reinforcing steel was included in some of the walls tested in flexure across a horizontal span. The performance of the walls laid in the various patterns is compared to that of standard running bond.

■ MASONRY CONSTRUCTION IS one of the oldest building techniques, and it survives today because of its beauty and its adaptability to form and architectural treatment. In the past, exposed masonry was confined to various kinds and sizes of brick and stone. Concrete block has been used as a backup material for many decades, but it is being used more and more as a finished wall material for both exteriors and interiors of homes, churches, schools, and public and commercial buildings. This popularity has been achieved through the low cost of construction and the adaptability of concrete masonry to a wide variety of interesting patterns.[1,2]

Block arranged in any desired pattern may of course be used for non-load-bearing walls, but patterns in load-bearing walls have been restricted by building code requirements or by the lack of information needed by design engineers and architects. The structural strength and stability of load-bearing walls constructed of 8 x 8 x 16-in. units laid in running bond (Fig. 1) have been established experimentally,[3,4] but there is little published information describing suitability of concrete masonry walls laid in other patterns.

To supply some of the needed data, the Research and Development Laboratories of the Portland Cement Association have carried out tests to determine the compressive and flexural strength of concrete block wall panels laid up in nine different patterns (Fig. 1). About 100 wall panels, 4 x 8 ft, were tested. The patterns were chosen as typifying several arrangements, and it is believed that the information developed from the selected patterns is applicable to any pattern that has been proposed.

TEST PROGRAM

The program was designed to investigate the structural properties of concrete masonry walls laid up in various block arrangements as compared to the already accepted load-bearing walls laid up in the standard running bond. The results were to form the basis for an evaluation of the various masonry wall patterns with respect to meeting code requirements for load-bearing structural walls. The objectives of the testing program were:

1. Determine the relative compressive strength of the various masonry patterns compared to the conventional running bond pattern in wall panels (4 x 8 ft)
2. Determine the flexural strength of walls of the same patterns and size with a uniform transverse load across a vertical span (reaction members at top and bottom)

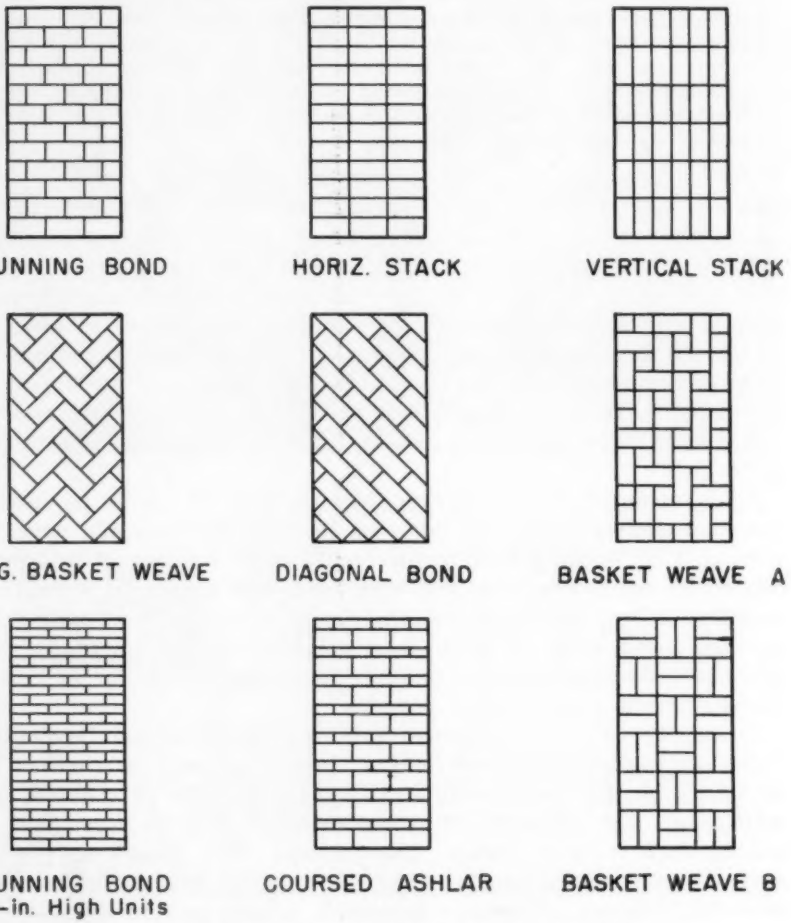

Fig. 1—Concrete masonry patterns for structural tests

ACI member **Richard O. Hedstrom** is development engineer, Products and Applications Development Section, Portland Cement Association Research and Development Laboratories, Skokie, Ill. Previous to joining PCA in 1955 he worked in the Engineering Department of Robert Hunt Co. He is a member of Committee 331, Structures of Concrete Masonry Units.

3. Determine the flexural strength of a limited number of patterns with a uniform transverse load across a horizontal span (reaction members at both ends)
4. Determine the effect of horizontal joint reinforcement on the transverse strength of walls across a horizontal span
5. Check the effect of a superimposed compressive load on the transverse load resistance of wall panels, with the transverse load applied across the vertical span of the wall panels
6. Investigate the effect of various mortars on wall strength

The tests were planned primarily to furnish information on the behavior under compressive and transverse loads of wall panels of sufficient size to represent story-height building construction. To achieve this, all compression and vertical span transverse load tests were carried out in accordance with ASTM E 72-55.[5] The following schedule indicates the architectural patterns included in these tests.

Series I—Compression tests

Compression tests were made on the following patterns which are illustrated in Fig. 1:

1. Running bond (8 x 8 x 16-in. units)
2. Horizontal stacked bond (8 x 8 x 16-in. units)
3. Vertical stacked bond (8 x 8 x 16-in. units)
4. Diagonal running bond (8 x 8 x 16-in. units)
5. Diagonal basket weave (8 x 8 x 16-in. units)
6. Basket weave-A (8 x 8 x 16, 8 x 8 x 8-in. units)
7. Basket weave-B (8 x 8 x 16-in. units)
8. Coursed ashlar (8 x 8 x 16, 8 x 4 x 16-in. units)
9. Running bond (8 x 4 x 16-in. units)

The concrete masonry units were commercial three-core stretcher units made with expanded shale aggregate, and cured with steam at atmospheric pressure. The walls were erected with Type M and Type S portland cement-lime mortars (ASTM C 270) using only face shell mortar bedding.

Series II—Transverse tests (vertical span)

Same units, mortar bedding, mortars and patterns as in Series I. Six of the walls used in this series of tests were repaired after failure and were then subjected to a combined compression and transverse load test.

Series III—Transverse tests (horizontal span)

Same units, mortars and mortar bedding as in Series I and II. Also some of these walls were reinforced with steel in the horizontal mortar joints. Wall patterns included in this series were:

1. Running bond (8 x 8 x 16-in. units)
2. Horizontal stacked bond (8 x 8 x 16-in. units)
3. Diagonal basket weave (8 x 8 x 16-in. units)
4. Running bond (8 x 4 x 16-in. units)

Auxiliary tests

Auxiliary tests included in the program to provide required information about the wall components and materials included the following:

1. Moisture content of units
2. Compressive strength of units
3. Mortar strength in compression and tension

ERECTION OF WALLS

Series I and II

The walls for tests of compression and transverse load across a vertical span were erected in groups of 12 walls at a time. Each group consisted of two walls of each of six patterns. One wall of each of the six patterns was tested in compression and the six companion walls were tested in flexure. The wall panels were 4 x 8 ft for both series of tests, the only exception being the walls erected in the two diagonal patterns which were 3 ft 9 in. wide by 8 ft 5 in. high.

To provide a rigid base on which the walls could be built, transported, and tested, reinforced concrete pallets were used. The walls were all laid up by an experienced mason following his usual style of workmanship. The first courses were laid in a full mortar bed on the pallets. All joints were given a final tooling to form a concave mortar joint. It is believed that the workmanship used in laying up the walls is representative of well-supervised building construction in this locality (Fig. 2).

All the walls were air cured in the laboratory at an average temperature of 73 F, and were tested between the ages of 28 and 35 days. During these first two series of tests, the walls were subject to relative humidity variations between 38 and 65 percent.

Fig. 2—Pallets, scaffolding, and alignment frame used in wall erection

Series III

The walls erected for testing with a transverse load across the horizontal span were 8 ft long by 4 ft high, except that walls built with the diagonal pattern were 3 ft 9 in. high by 8 ft 5 in. long. The same type of units and mortars were used in this series as in Series I and II. Some of the walls were erected with two different amounts of horizontal reinforcing steel. The steel was a truss type of masonry reinforcement made with #9 wire.

The walls were erected on 6-in. steel channels and were not moved after erection. To reduce restraining friction at the bottom of the walls, the steel channels were covered with an envelope of polyethylene sheet containing a thin layer of fly ash. The walls were then erected on top of the plastic in

TABLE 1 — PHYSICAL PROPERTIES OF MORTARS AS USED IN WALLS

Type and mix proportions	Age, days	Strength, psi				
		Compression		Tension		Bond*
		Cure A†	Cure B‡	Cure A	Cure B	Cure B
Type M	3	1100	—	145	—	44
1:1¼:4¾	7	1660	—	200	—	52
C:L:S	14	2200	1780	240	200	58
	28	2500	1940	275	250	58
Type S	3	420	—	75	—	15
1:2:7	7	720	—	100	—	18
	14	950	900	130	120	22
	28	1290	1240	180	155	26

*Two block pier—stored with walls.
†Cure A—Moist cured 7 days followed by air storage at 50 percent relative humidity.
‡Cure B—Continuous storage at 50 percent relative humidity.

Note: The compressive strengths of these mortars mixed and cured according to ASTM C 270 were: Type M—4050 psi; Type S—2100 psi.

the same manner as the walls for Series I and II. All the walls in Series III were air cured a minimum of 14 days at 73 F and 50 percent relative humidity.

Combined load test

The wall panels used for the combined compression and transverse load test were walls that had been tested in Series II, with a transverse load across the vertical span and had been repaired with a polyester resin. In these walls the first test failure always occurred on one side of the mortar joint only, with the mortar apparently firmly bonded to the block on the opposite side. Without removing this mortar, the two pieces of the wall panel were glued back together with the resin. It was believed that this method of repair restored the walls to approximately their original strength.

TESTING PROCEDURES

Properties of mortars and units

The mortar mixes used in Series I and II were established on a strength basis and with the mason's consultation as to workability. The mason found the most severe demand on the mortar to be the requirement that it hold together on the 16 in. vertical face shell when erecting the vertical stacked bond wall. This resulted in establishing a Type M (ASTM C 270) mortar mix in the following proportions: 1 part portland cement, 1¼ parts pressure hydrated Type "S" lime, and 4¾ parts sand. The second mortar mix, designated as Type S mortar, had the following proportions: 1 part portland cement, 2 parts pressure hydrated Type "S" lime, and 7 parts sand. The actual proportioning was done by weight. The water content of the mix was adjusted to the mason's instructions and after being established the mixes were maintained at the desired consistency. Flow determinations were made on each batch according to ASTM C 109-56, "Method of Test for Compressive Strength of Hydraulic Cement Mortars."

ASTM C 270-57T, "Mortar for Unit Masonry," gives two methods of classification of the mortar types. These methods allow the mortar to be classified either by the *property* specification or the *proportion* specification, and for these tests the property specification was used. When made according to this specification a Type M mortar must have a minimum average compressive strength of 2500 psi at 28 days under continuous moist curing. A Type S mortar must have

TABLE 2 — COMPRESSIVE STRENGTH OF MASONRY WALLS

Wall pattern	Compressive load on wall at failure, lb		Primary failure	Compressive strength					Block‡ corrected	Ratio‡ wall to block percent	Safety§ factor
	Mortar type			Block*	Measured*	Adjusted†	Percent of standard				
	M	S					Individual	Average			
Runnning bond	233,700			1400	638	638	100		1400	45.6	7.5
		201,000	Block	1450	549	530		100	1450	38.9	6.5
		175,500		1165	480	576	100		1165	41.2	5.6
Horizontal stacked bond	236,250			1400	645	645	101		1400	46.1	7.6
		219,700	Block	1450	601	580	105	103	1450	41.4	7.1
Vertical stacked bond	161,100			1400	440	440	69		805[a]	54.7	5.2
		164,400	Block	1315	449	478	86	78	915[a]	49.1	5.2
Diagonal basket weave	176,700			1400	515	515	81		1100[b]	46.8	6.1
	143,750		Mortar	1165	419	503	79	75	910[b]	46.1	4.9
		131,000		1450	381	367	66		1200[b]	31.8	4.5
Diagonal running bond	150,200			1400	438	438	69		1100[b]	39.8	5.2
	146,200		Mortar	1165	426	512	80	76	910[b]	46.8	5.0
		139,000		1315	405	430	78		1115[b]	36.3	4.8
Basket weave A	205,400			1400	562	562	88		1100[c]	51.1	6.6
		129,500	Block	1450	354	341	62	74	1200[c]	29.5	4.2
Basket weave B	163,000			1450	446	430	67		1120[c]	39.8	5.2
	170,200			1450	455	438	69		1120[c]	40.6	5.4
	149,000		Block	1320	407	432	68	77	825[c]	49.4	4.8
		190,000		1315	519	551	100		1050[c]	49.4	6.1
		156,250		1315	427	454	82		1050[c]	40.7	5.0
Coursed ashlar	187,250			1450	512	554	87		1295[d]	39.6	6.0
	176,500		Block	1165	482	580	89		1165	41.3	5.7
		178,000		1315	486	554	100	95	1225[d]	39.6	5.7
		179,500		1165	491	590	106		1165	42.1	5.8
4 in. Running bond	238,750			1450	654	670	105		1365[d]	47.9	7.7
		201,250	Block	1315	550	628	113	109	1225[d]	44.9	6.5

*Gross area, psi

†Adjusted to block strength of 1400 psi

‡Corrected for position of block in wall

[a] Average unit compressive strength when tested in vertical position (as used in vertical stacked bond patterned walls)

[b] Since units are laid at a 45 deg angle the assumed unit strength used is the average of the strengths in the horizontal and vertical direction

[c] Assumed unit strength = $\dfrac{f_{e1}A_1 + f_{e2}A_2}{(A_1 + A_2)}$

f_{e1} = unit strength (horizontal position)

f_{e2} = unit strength (vertical position)

$A = A_1 + A_2$ = Area of weakest cross section of wall (cross section containing highest percentage of vertical units)

A_1 = that part of A containing horizontal units

A_2 = that part of A containing vertical units

[d] Unit strength of 4-in. high units

§Safety factor = $\dfrac{\text{Wall strength}}{\text{Wall allowable load (85 psi)}}$

a strength of at least 1800 psi at 28 days. The mortars as used and designated in these tests had average 28 day compressive strengths of 4050 psi for Type M mortar and 2100 psi for Type S mortar under continuous moist curing. It should be recognized that mortars complying with this specification are made with a different flow and are cured under different conditions than those used in construction of the walls.

The proportions of the mortar mix were based on surface-dry, loose volumes of sand. The mortar sand was purchased locally and was found to have a uniform grading. The average gradation was:

U.S. Standard Sieve No.	Percent retained
4	0
8	Trace
16	9
30	34
50	84
100	98
Fineness modulus	2.25

Cubes and briquets were made from the batches of mortar used by the mason. Some of these specimens were air cured with the walls and some were fog cured for 7 days. The physical properties of the mortars are given in Table 1.

The two types of curing of the mortar cubes and briquets reported in Table 1 were chosen to provide a more realistic estimate of the strength of mortar in the wall at the time of test. Other tests have indicated that the strength of mortar moist cured for 7 days is somewhat greater than that of mortar in the wall while the strength of mortar cast in nonabsorptive molds and cured continuously at 50 percent relative humidity is slightly weaker.

All block were artificially dried and then stored in the laboratory for several weeks so that they were approximately in moisture equilibrium with the ambient atmosphere at the time the walls were erected. Sufficient units were taken from each lot of block to determine the physical properties of the block under ASTM C 90-52, "Standard Specification for Hollow Load-Bearing Concrete Masonry Units." The absorption ranged from 15.3 to 17.1 percent for the 8-in. high units and 20.3 to 21.3 percent for the 4-in. high units. The net area of the 8-in. units was 57.1 percent and for the 4-in. units, 55.7 percent. The weight of the concrete in the units averaged 79.2 lb per cu ft. Compressive strength of the units and the bond strength of two-unit couplets stored with the walls were determined at the time the walls were tested. The compression tests on the units were made in accordance with the methods of ASTM C 140-56, "Sampling and Testing Concrete Masonry Units," and results are recorded in Table 2. The tensile bond strength was determined by the method illustrated in Fig. 3, in which a moment was applied to a two-unit couplet by loading a lever arm clamped to the top unit, an adaptation of a method previously used by Saemann[6] and others. Tensile bond strength was determined by the formula

$$S = \frac{LD}{CA}$$

S = average tensile bond strength, psi
L = applied load, lb (including equivalent weight due to lever arm dead weight)
D = lever arm, in.
C = distance between center lines of face shells, in.
A = mortar contact area on tension face shell, sq in.

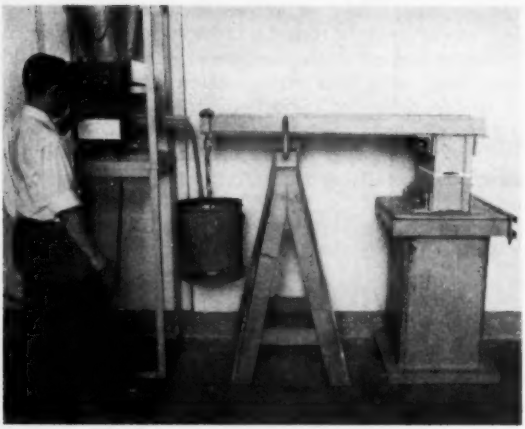

Fig. 3—Apparatus used to test two-block couplet for mortar bond strength

Compression testing of walls

All of the walls tested in compression were capped with a neat cement paste and were loaded through a 2-in. steel plate. The tests were in accordance with ASTM E72-55. An eccentric load was applied through a spherical bearing block and 21 in. wide-flange beam, with web stiffening plates, to a 1-in. sq cold-rolled steel rod placed one-sixth the wall thickness from the central plane of the wall. The bearing areas on both sides of the rod were cushioned with leather belting.

Wall deformations were measured by four vertical compressometers of 88-in. gage length, two on each side of the wall. Five dial gages were mounted on each end of the wall in a special frame for measuring the transverse deflections.

The walls were tested by a succession of loading and unloading operations with progressively increasing loads. Compressometer and transverse deflection readings were taken after each load was applied, and the amount of set was recorded when the load was released.

Transverse testing of walls

Two types of transverse load tests were performed. The transverse strength of a vertical span was determined as specified in ASTM E 72-55 in a special testing machine designed for the purpose (Fig. 4). The test walls were built in the same manner as the compression test walls, and when ready for testing were set on a steel plate which rested on two 1-in. diameter steel rollers. The transverse load was applied uniformly over a 7 ft 6 in. vertical span by an air-filled plastic bag.

The transverse strength of a horizontal span was determined by the same testing apparatus used in the vertical-span transverse load tests except that the plastic bag and frame were rotated 90 deg and the bearing members were attached to the two ends of the wall instead of at the top and bottom.

The transverse load was applied in equal increments. After measuring the deflections at each increment, the load was released and the set was recorded.

In the vertical-span transverse load tests the deflections were measured at both ends of the walls. In the horizontal-span transverse load tests the deflections were measured at the top, bottom, and center of the walls at midspan.

Combined load testing of walls

The six walls tested with a combined static compression load and increasing transverse loads, which had been tested previously in flexure across a vertical span and had been repaired with a polyester resin, were placed in the flexural test apparatus so that the side that had been the compression face in the first test was now the tensile face. The compressive load was centrally applied across the 4 ft length of the wall and was equal to the usual code allowable load of 85 psi of gross wall area. The lateral load was then applied in increments of 20 lb per sq ft to the face of the wall. The horizontal deflections under load were recorded and the set after release of the load was recorded after each increment.

TEST RESULTS

In the following discussion of the test results, data are presented which are most pertinent to the load-carrying capacity of the walls. All measurements are on file and are available, but data which appear to be of secondary importance have been omitted.

Mortar

The physical properties of the mortars are recorded in Table 1 and show that moist curing provides a considerable improvement in strength. This is particularly true of Type M mortar, which contains a higher percentage of portland cement. The wall tests were conducted under conditions that were relatively unfavorable in developing mortar strength. The block and walls were stored continuously at approximately 50

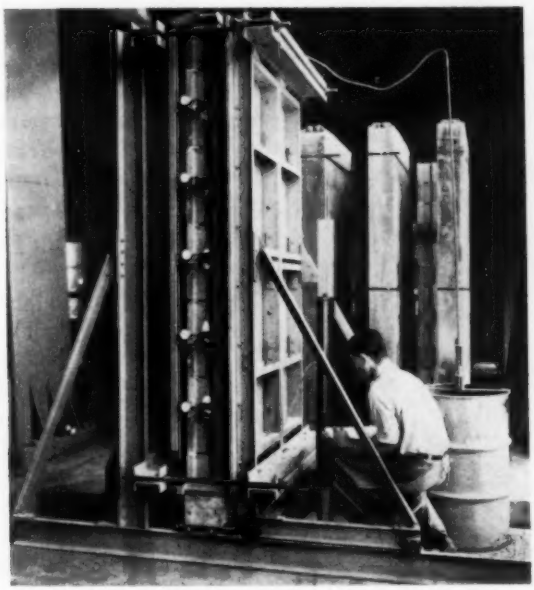

Fig. 4—Vertical span transverse load testing apparatus

percent relative humidity so that the only water available for hydration of the cement was that initially contained in the mortar. The initial absorption of water from the mortar tends to increase the strength by lowering the water-cement ratio; however, since much of the water absorbed by the block is quickly lost to the atmosphere, the mortar does not receive the benefit of further curing.

Compression tests

The compressive strength data for all walls are summarized in Table 2. The running bond pattern has become an accepted standard, and the performance of the various wall patterns is compared to that of the running bond. Over 2 years block obtained in various shipments from the same plant showed a range in compressive strength from 1165 to 1450 psi and on that account a direct comparison of the ultimate strength does not provide a true comparison of the relative strengths of the various walls. The column in Table 2 that expresses the wall strength as a percentage of the standard includes a correction for the difference in compressive strength of the units. This particular correction is based on the direct compressive strength of the unit as normally tested.

There was little difference in compressive strength of walls in which all block were laid horizontally, i.e., the 4- and 8-in. running bond, horizontal stacked bond, and coursed ashlar. The strength of walls containing diagonal or vertical block was about 75 percent of the standard. The strength of the individual block tested in a vertical direction (i.e., loaded on the ends) was 60 to 70 percent of the strength obtained when tested in a horizontal direction, and this was reflected in the lower strength of walls containing vertical or diagonal block. The strength of the mortar did not appear to have a marked effect on the compressive strength of the wall.

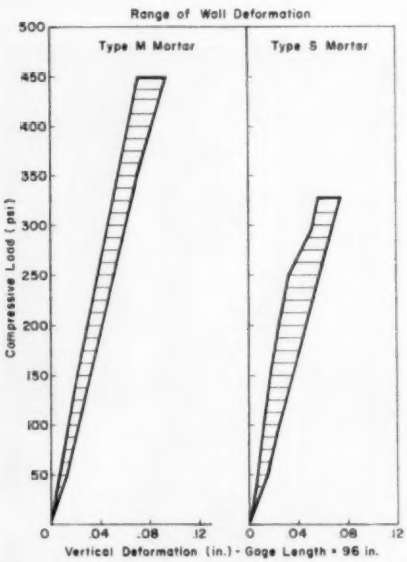

Fig. 5 — Load-strain relationship of walls, nine different bond patterns

The values expressing wall strengths as a percentage of the standard provide realistic comparisons for all walls that failed because of failure of the units, but may not be applicable to diagonal patterns with weak mortars that fail in shear. The results do not show the large reduction in strength that probably would be obtained with the diagonal pattern

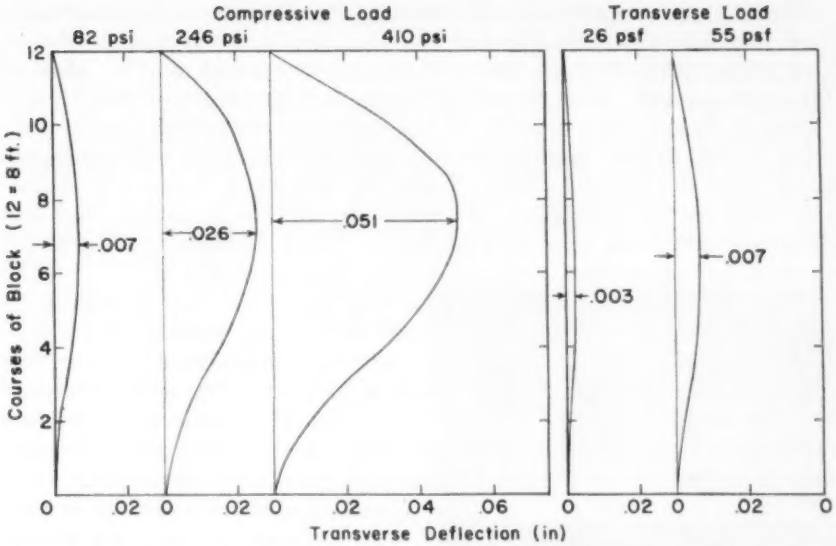

Fig. 6—Deflections of 8 in. masonry wall running bond pattern, Type M mortar

if the mortar strength was near the lower limit of the ASTM specification for a mortar to be used for load bearing walls (750 psi). Even with the relatively high strength mortars used, the failures in the diagonal patterns built using Type S mortar were primarily by shear along the joints, rather than in the block. With the high strength mortar, however, this shear failure occurred at loads close to the ultimate strength of the block and probably would be materially lowered as the strength of the mortar was reduced.

Other investigators have compared the compressive strength of walls to the strength of individual units, and this comparison is also given in Table 2; however, the comparison is based on the strength of the unit as used in the wall, i.e., horizontal, vertical, or diagonal. With four exceptions, all walls failed at unit loads of 35 to 50 percent of the strength of the individual block. Two of the four exceptions showed strengths greater than 50 percent of the block strength, and two showed less than 35 percent.

Fig. 5 shows, for all nine patterns, the total range of vertical deformation adjusted from the 88-in. gage length to the 96-in. gage length provided in ASTM E 72-55. The vertical deformation of the individual faces of the walls was apparently sensitive to precise location of the 1-in. loading bar and the alignment of the wall and testing equipment. However, the limited range shown by Fig. 5 indicates that the average deformation of the individual walls was not greatly affected by wall pattern or type of mortar.

The transverse deflection of walls under an eccentric compressive load also appeared to be affected more by the precise alignment of wall and testing setup than by the wall pattern or type of mortar. Most of the walls showed the expected deflection and curvature similar to that shown in Fig. 6. The rotation of the wall was restrained considerably more at the bottom than at the top, and this produced unsymmetrical bending of the wall. At lower loads the maximum transverse deflection occurred at midheight, but gradually moved up as the load and deflection increased, as shown in Fig. 6. The maximum transverse deflection for any of the walls tested was less than 0.09 in.

Four of the walls deflected under eccentric load into a slightly S-shape. This was probably due to some small misalignment of the wall or testing setup, but this was not definitely established. Under this condition the dial at midheight indicated practically zero deflection throughout the test. Reruns of these particular walls produced the deflection curves normally expected, and approximately the same ultimate strengths were obtained. Since all transverse deflections were small, they had little effect on ultimate strength of the wall panels.

Fig. 7 — Vertical stacked bond wall with Type S mortar at moment of compression test failure

The running bond walls with Type M mortar developed over 45 percent of the unit strength at failure and those with Type S mortar developed about 40 percent. Actual failure in every case occurred in the top two courses of units which failed in compression of the face shell carrying the greater load. Compression failure was accompanied by a splitting of the webs from the top unit down through at least three courses.

The horizontal stacked bond wall erected with the Type M mortar developed 46 percent of the unit strength at failure, and the wall with Type S mortar developed 41 percent. The failures duplicated those of the running bond. Before ultimate failure of the wall built with Type S mortar, however, bond failures were observed along the continuous vertical joints. One joint opened at a compressive load of 110 psi and the other at 230 psi.

The shortening of the horizontal stacked bond walls under the compression loading was of the same magnitude as the shortening of the running bond walls. The amount of transverse deflection, however, was less for the horizontal stacked bond walls than for the running bond patterned walls. This was in accord with the results, discussed later, that were obtained from the vertical span transverse load tests.

The vertical stacked bond walls reflected the reduced compressive strength of the units when laid on end. Although the walls developed about 50 percent of the strength of the units, based on tests in the vertical position, the units themselves were only 60 percent as strong when tested in this way as when tested in the horizontal position.

The primary failure of the walls erected in the vertical stacked bond pattern was not a compression failure of the webs. After the webs split, the face shells buckled and collapsed (Fig. 7). The web failure started at midheight of the wall and spread both upward to the top course of units and downward to the first course of units before the face shells buckled. This failure pattern was the same with both the Type M and S mortars. The wall erected with the Type M mortar developed about 5 percent more of the unit strength than did the wall erected with the Type S mortar.

The diagonal basket weave patterned walls showed failures under compression loading which were more complex than those of the other walls. With these walls the type of mortar proved to be important. With the Type M mortar the wall strength was 46 percent of the unit strength (see footnote ‡, Table 2), and primary failure was a web failure of the units followed by compression failure and buckling of

Fig. 8 — Diagonal basket weave wall with Type S mortar after compression test failure

the face shells. Some secondary shear failure was also observed, although there was no indication that with this mortar the shear failure was involved in the initial failing of the wall. With the Type S mortar the wall strength was 32 percent of the unit strength, and the failure was due entirely to a shear failure along a diagonal mortar joint and through the unit which broke the continuity of the mortar line (Fig. 8). With weaker mortars even lower strengths would be expected.

The results obtained on the diagonal running bond walls were similar to those obtained on the diagonal basket weave walls. With these walls, though, the primary cause of failure with both types of mortar was a shear failure. With the Type M mortar, the units in the walls were at stresses close to ultimate because, accompanying the shear failure, the webs of the units above the shear line split and the face shells failed at least partially in compression. A vertical tension crack also developed from the shear line down through at least the next four courses at the midwidth of the wall.

Fig. 9—Basket weave wall built with Type S mortar at moment of compression test failure

In basket weave patterns A and B the walls were erected with a combination of horizontally and vertically placed units. The vertically placed units, being weaker, controlled the strength of the wall. With both patterns and both mortars the vertically placed block failed in tension between the face shells and the webs, followed by buckling of the vertical face shells and splitting of the webs of the adjacent horizontal units (Fig. 9). The two types of basket weave patterns built with the Type M mortar averaged about 45 percent of the calculated unit strength (see footnote ‡, Table 2). The same patterned walls built with the Type S mortar averaged almost 40 percent of the calculated unit strength. These percentages were about the same as those obtained with the running bond walls, although with the basket weave walls the calculated unit strengths were lower and the wall strengths were proportionally lower also.

The last two wall patterns involved in this series of tests both used the nominal 4-in. high unit. The first of these two patterns, the coursed ashlar, used alternate courses of 4-in. units and standard 8-in. nominal height units. The testing of this wall pattern showed that the weaker units, either the 4- or 8-in. units, determined the wall strength, and also that when used in combination the weaker units always failed by splitting of the webs from the face shells. After the webs failed in the weaker units the web failure spread to the stronger units and the face shells then buckled and collapsed. With the coursed ashlar patterned walls, differences in the two types of mortar appeared to have little effect on the compressive strength of the walls. The walls using the coursed ashlar bond pattern developed, on the average, slightly over 40 percent of the strength of the weaker units whether erected with Type M or S mortars.

Results obtained on the running bond walls erected with the nominal 4-in. units generally were similar to those obtained on walls erected from the nominal 8-in. units. Because the 4-in. units had thinner face shells and webs than the 8-in. units it was evident that the tensile strength of the concrete was quite important. Units which had compressive strengths of less than 1250 psi of gross area failed in a wall by splitting of the webs from the face shells, and not as a compressive failure of the face shells.

TRANSVERSE LOAD TESTS

Transverse loads across a vertical span

In this series all nine wall patterns were tested with both strengths of mortar, and results showed that the transverse strength was primarily dependent on the bond strength obtained between the mortar and units. All data are recorded in Table 3, including the relative strengths of the walls compared to the standard running bond. All patterns with continuous horizontal mortar joints failed in bond between unit and mortar along one of the horizontal mortar joints at approximately mid-height of the wall. The diagonal patterns also failed in bond, but the failure traversed a saw-tooth line across the width of the wall, following the mortar joints. The diagonal patterns developed strengths about 50 percent higher than those of the running bond, as might be expected because of the added bond area and the angular displacement of the tensile forces along the mortar lines.

The horizontal stacked bond walls developed transverse strengths which were higher than those obtained for walls erected with the running bond pattern. This may be explained as the result of continuity of the vertical mortar joints. The tensile strength of the mortar was much greater than the bond strength of the mortar to the units, and well-filled vertical mortar joints thus effectively increased the flexural

TABLE 3 — TRANSVERSE STRENGTH OF MASONRY WALLS

Wall pattern	Mortar type	Vertical span transverse strength, lb per sq ft				Horizontal span transverse strength				
		No compressive load			Compressive load, 85 psi	Unreinforced			Horizontal reinforcement, lb per sq ft	
		lb per sq ft	Percent of standard	Average*		lb per sq ft	Percent of standard	Average*	16 in. c-c	8 in. c-c
8 in. running bond (standard)	M	60.0	100	100	425.0	127.0 136.0	100		149.4	203.0
	S	34.7 32.2	100			127.0 123.0	100	100	149.9	202.3
Horizontal stacked bond	M	85.0	141	130	405.0	47.7	37	28	130.0	191.2
	S	39.9	120			29.2	24		131.3	190.0
Vertical stacked bond	M	60.6	101	87	357.5					
	S	24.6	74							
Diagonal basket weave	M	89.0	148	151	410.5	69.5 76.9	56 67	60		
	S	51.7	155			65.8	57			
Diagonal running bond	M	103.0	172	158	429.0					
	S	48.1	144							
Basket weave A	M	69.9	117	105	400.0					
	S	30.7	92							
Basket weave B	M	42.9 44.5	72 72	72						
	S	24.4	73							
Coursed ashlar	M	43.3 55.5	72 93	83						
	S	26.3 34.7	79 104							
4 in. running bond	M	65.0	108	101		169.4 173.0	129 133	130	160.0 193.0	196.0
	S	31.3	94			158.0	128		186.6	194.7

*Average for Type M and S mortars

strength of the walls. This continuity of the vertical mortar joints also stiffened the walls in the compressive tests, as shown by the reduced transverse deflections of the horizontal stacked bond pattern as compared with the running bond. The difference between the bond strength and tensile strength of the mortars was also demonstrated in the failure pattern of most of the walls tested. With the running bond pattern made with both the 4 in. and 8 in. high units, and the coursed ashlar patterns, the location of horizontal bond failures was between mortar and units on the side of the mortar joint opposite a vertical joint.

For walls with straight horizontal joints, closer spacing of the joints developed more uniform bending and higher flexural strengths. Although there was little difference in the flexural strength of walls with joint spacings of 4 or 8 in., there was an appreciable loss in strength when joint spacing was increased to 16 in. Combinations of the effects of horizontal joint spacing and continuity of vertical mortar joints greatly account for the wide range of recorded data.

For the running bond walls, the values of transverse strength (lb per sq ft) recorded in Table 3, are, for the test setup and span used, nearly equal to the computed extreme fiber stresses, and hence to the bond strength of the mortar (psi). These values indicate that the bond strength of the Type M and S mortars were about 56 psi and 32 psi, respectively. These compare favorably with corresponding bond strengths of 58 psi and 26 psi recorded in Table 1 for two-block piers.

Transverse loads across a horizontal span

This series of tests included four wall patterns with three mortars and with various amounts of reinforcement in the horizontal joints. The horizontal stacked bond was chosen as representative of patterns depending entirely on mortar bond for transverse strength. Other patterns included the diagonal basket weave and the 4 and 8 in. running bonds which developed lateral strength through shear strength in the joints between interlocking units. The transverse strengths of all walls are recorded in Table 3.

From a comparison of the curves of Fig. 10 and transverse strengths recorded in Table 3, it is evident that of the unreinforced walls, those with the running bond pattern were much stronger than the others, and that the unreinforced running bond walls built with the two mortars showed little difference in transverse load resistance across the horizontal span. The transverse strength of the horizontal stacked bond and diagonal basket weave patterns were about 30 and 60 percent of the standard, respectively. The running bond wall with 4-in. units was about 30 percent stronger than the standard wall with 8-in. units.

The addition of reinforcement steel in the 8 in. running bond walls, either in every course or in every other course, had little effect on the stiffness, or transverse deflections, for loads up to about 80 lb per

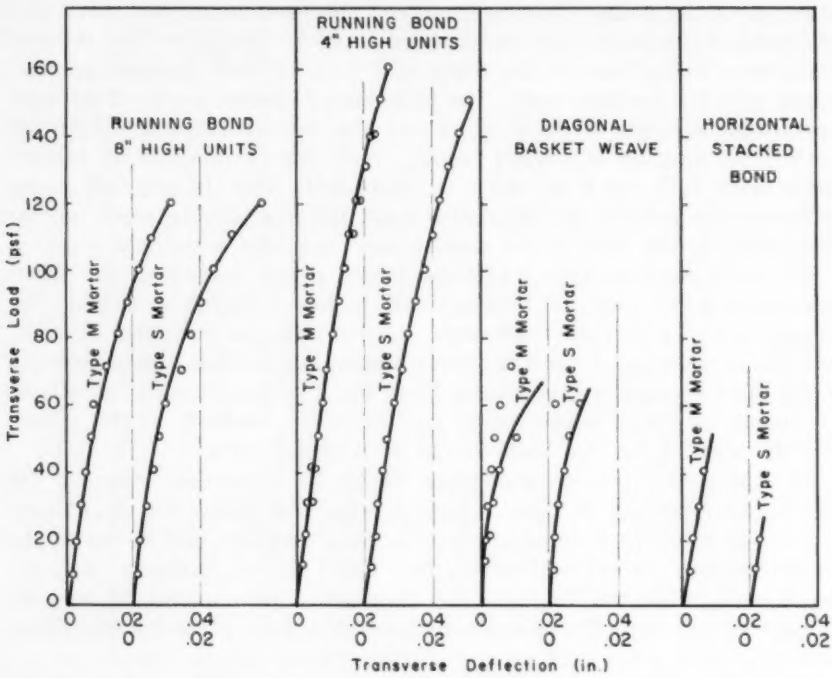

Fig. 10—Horizontal span transverse load test deflection (at midspan), no horizontal steel

sq ft. Reinforcement in every other course (16 in. c-c) showed only a minor effect on the ultimate strength as compared to that of the unreinforced walls, but reinforcement in every course increased the ultimate strength by 55 to 70 percent. All walls failed with a straight vertical crack passing alternately through the joints and the block near the center of the span.

The unreinforced horizontal stacked bond walls were the weakest walls tested with the lateral load applied across the horizontal span. The strength of the walls was a little less than 50 lb per sq ft when built using Type M mortar, and was about 30 lb per sq ft with the Type S mortars. However, with reinforcing steel in every other course, the transverse strength was increased to about 130 lb per sq ft (Fig. 11) which was equal to 87 percent of the strength of the running bond walls with the same amount of reinforcing steel. With horizontal steel in every course, or 8 in. on center, the strength of the stacked bond pattern was 94 percent of the strength of the running bond walls with the same amount of steel. These figures are true only of the ultimate loads, which were primarily a direct function of the area of reinforcing steel in the walls. The reinforcing steel did little to increase the load

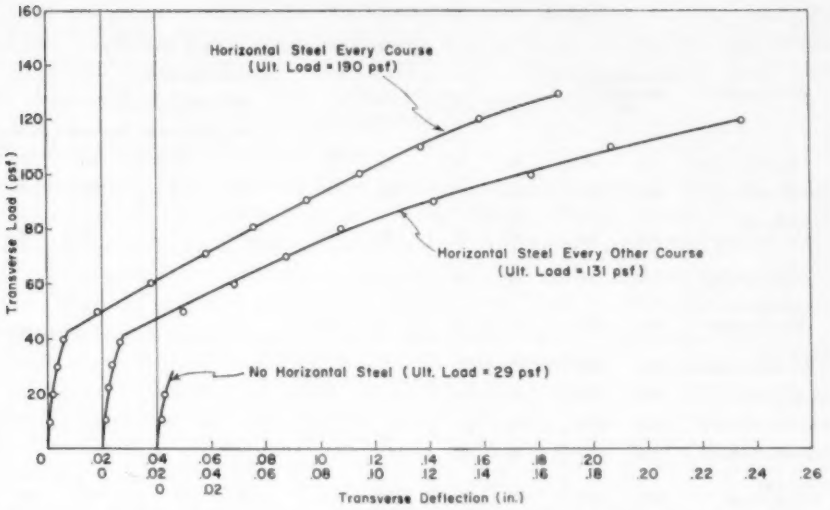
Horizontal Steel Every Course
(Ult. Load = 190 psf)
Horizontal Steel Every Other Course
(Ult. Load = 131 psf)
No Horizontal Steel (Ult. Load = 29 psf)
Transverse Load (psf)
Transverse Deflection (in.)

Fig. 11—Horizontal span transverse load test deflection (at midspan), horizontal stacked bond pattern, Type S mortar

at which the first cracking appeared when using the Type M mortar, but with the Type S mortar the first cracks appeared in the unreinforced walls at about 40 percent lower loads than for the reinforced walls built with this mortar. In all of the stacked bond patterned walls, failure was always in bond between the mortar and the units, with the steel failing at the same location.

The running bond walls built with the nominal 4-in. units proved to be the strongest unreinforced wall pattern in the transverse load tests, with strengths about 30 percent greater than those of running bond walls built with the nominal 8 in. high units. This increased strength was also observed when horizontal reinforcing steel was placed 16 in. c-c with both mortars, and averaged about 20 percent higher than for the similarly reinforced running bond walls built with the 8 in. high units. When the horizontal reinforcing steel was placed 8 in. c-c the walls had the same ultimate strength as the running bond walls built with the 8 in. high units and the same amount of reinforcement.

Transverse loads across a vertical span with a superimposed compression load on wall

Six walls that had been tested across the vertical span with transverse loads, after being repaired by a polyester resin adhesive, were retested with transverse loads across the vertical span as before, but with an 85 psi compressive load applied centrally along the top of the walls. This was done to check the effect of the compressive load on the

TABLE 4 — SUMMARY OF RESULTS

Wall pattern	Compressive strength,* psi		Transverse strength, lb per sq ft							
			Vertical span		Horizontal span					
					Unreinforced		Reinforced horizontally			
							Spaced 16 in. c-c		Spaced 8 in. c-c	
Mortar type	M	S	M	S	M	S	M	S	M	S
Running bond	640	550	60	35	130	125	150	150	205	200
Horizontal stacked bond	645	580	85	40	50	30	130	130	190	190
Vertical stacked bond	440	480	60	25						
Diagonal basket weave	510	365	90	50	75	65				
Diagonal running bond	475	430	105	50						
Basket weave A	560	340	70	30						
Basket weave B	435	500	45	25						
Coursed ashlar	570	570	50	30						
4 in. running bond	670	630	65	30	170	160	175	185	195	195
Average	550	495	70	35	105	95	150	155	195	195

*Compression results adjusted to 1400 psi unit in wall.

transverse load resistance of the walls. A uniform compressive load of 85 psi on a wall will produce a compressive force on the face shell mortar joints equal to 260 psi. By the standard flexural formula, this compressive force plus the bond strength of the mortar joint (both in psi) should be equal to a factor of 1.02 times the uniform transverse load on these walls (lb per sq ft) across a 7½-ft span. The tensile bond strength averaged about 60 psi. The transverse loads at first observed cracking ranged from 289 to 336 lb per sq ft for the six walls tested. The test results agreed well with the calculations for all six walls tested. After first cracking the wall continued to carry increasing loads until the transverse load had caused the wall to bend so far that the compressive load could no longer be sustained and the walls collapsed. The ultimate loads, as given in Table 3, ranged from 357 lb per sq ft for the vertical stacked bond patterned wall to 427 lb per sq ft for the diagonal running bond patterned wall.

SUMMARY

Compressive strength

The compressive strength of walls in which all block are laid in a horizontal position is about one-third greater than that of walls containing similar block in diagonal or vertical positions. However, the weakest wall in this series had a safety factor of 4.2, based on an allowable wall load of 85 psi, so it appears that any of the patterns tested would be satisfactory for load-bearing walls.

The compressive strength of a wall depends primarily on the strength of the block and is little affected by the strength of the mortar, with the possible exception of walls with diagonal patterns which may fail in shear along the diagonals if built with weak mortar.

Flexural strength—Vertical span

Failure of all walls tested for flexural strength over a vertical span was in the bond of the mortar to the block. Walls constructed with Type M and Type S mortars showed mortar bond strengths of 60 and 30 psi, respectively, which agreed well with tests of two-block piers. The diagonal patterns were about 50 percent stronger than the standard running bond, due to the longer saw-tooth line of failure. For other patterns, continuous vertical mortar joints appeared to strengthen the walls, and patterns with wider horizontal joint spacing, as with vertical block, appeared to weaken the walls. Due to combinations of these effects flexural strengths varied from 72 to 130 percent of the standard running bond.

Flexural strength—Horizontal span

The block pattern exhibited a marked effect on the flexural strength of walls tested over a horizontal span. Running bond patterns were considerably stronger than the others, due to the interlocking action of the units, and the running bond walls with 4 in. high units were about 30 percent stronger than those with 8 in. high units. With the running bond patterns the flexural strengths were affected only slightly by differences in the two mortars. Walls with continuous vertical joints, depending entirely on mortar bond for transverse strength, were about 30 percent as strong as the standard running bond.

The particular joint reinforcement used did not materially increase the load at first cracking, but it was effective in increasing the ultimate strength of the walls. Joint reinforcement was particularly beneficial in walls with continuous vertical joints.

Combined load tests

The addition of a vertical compressive load to the walls tested in flexure across a vertical span proved to be an effective method of increasing the flexural strength. These tests showed that the utilization of the wall's load carrying capacity was one way of increasing the stability of the wall to transverse loads.

Effect of mortar type

The compressive strength of masonry walls laid up with Type M mortar was only 10 percent greater than that of walls with Type S mortar, showing that the type of mortar (M or S) is relatively unimportant with respect to compressive strength of walls. The horizontal

span flexure tests with both reinforced and unreinforced walls, with the exception of the stacked bond pattern, also showed little if any difference due to mortar type. The flexural strength of the stacked bond pattern over a horizontal span without horizontal reinforcement was entirely dependent on the bond strength of the mortar; and walls with Type M mortar were considerably stronger than those with Type S mortar. Likewise, in the vertical span flexure test, the walls with Type M mortar were twice as strong as those with Type S mortar, reflecting the higher bond strength of this Type M mortar.

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Water-Cement Ratio Versus Strength—Another Look

By HERBERT J. GILKEY

The water-cement ratio (W/C) pronouncement probably marked the most useful and significant advance in the history of concrete technology.

From the beginning, however, there have been dissenters who in their tests or research have happened to touch areas of unusual gradings or areas that entailed comparisons between mortars and concretes or between neat cement pastes and sand-cement mortars.

Besides the actual dissenters there have been thoughtful operators in the area of large-aggregate concrete, used regularly in dams, who, recognizing the lack of information on possible effects of large aggregates and/or large specimens on strength have serious doubts as to whether or not the mass concrete in the structure would develop the strength that the W/C relationship has allocated to it.

With current attention being redirected toward possible limitations in the W/C generalization, now may be the time to exhume and pull together scattered pertinent evidence that has, bit-by-bit over the years been presented, and forthwith become buried in the voluminous literature of concrete. The aim is not to discredit the water-cement ratio as a useful empiricism but rather to focus attention on both its range of applicability and on its limitations.

The paper calls attention to, and discusses briefly, a number of the published allegations of invalidity, indicating some of the pros and cons brought out in discussions thereof. As support for tentative explanations, pertinent stress-strain and water-gain data are presented. Finally a modified, duly restricted and qualified version of a W/C versus strength relationship is proposed.

■ THE WATER-CEMENT RATIO (or voids-cement ratio—essentially identical) versus strength relationship had been recognized for mortars by the French investigator Feret 20 years or more prior to the Abrams pronouncement and apparently there were other independent discoveries. Its active promotion and growing importance in concrete technology, however, both in this country and abroad, date from the work of Abrams,[1] probably the best known and most influential publication in the history of concrete. It was Abrams who performed and published extensive background tests and formulated statements that have for many concrete practitioners, elevated the ratio of water to cement, or cement to water, to the near-sacred status of "the law of proportioning for concrete." For all investigators the role accorded to the aggregate was that of inert filler; a diluent that decreased the cement requirement.

Herbert J. Gilkey, professor and formerly head of the Department of Theoretical and Applied Mechanics, Iowa State University, Ames, is a past president of ACI (1949) and has long been active in Institute affairs, both on technical and administrative committees. An accepted authority in the field of research in engineering materials, especially concrete, he is the author of over 150 technical papers and discussions many of which have appeared in the ACI Journal. Professor Gilkey is currently a member of ACI Committee 115, Research.

The importance of the W/C concept cannot be overrated despite the fact that the pronouncement seems definitely to have constituted an unfortunate overstatement. Because of the unquestioning near-universal acceptance of the pronouncement in its unqualified entirety, doors-of-doubt that should have been left ajar were closed. Aspects that should, during the elapsed years, have been receiving critical scrutiny were accepted as being not pertinent, i.e., finished business. The major contribution of the W/C concept was as the term implies, the focusing of attention on the importance of the ratio of the mixing water to the cement. The luster of that contribution stands undimmed. The weakness, the disservice to the cause of concrete lies, not in what the pronouncement as made included, but rather in what it excluded on the basis of assumed nonrelevance.

Abram's initial statement appears in his discussion of Fig. 1, Reference 1 (italics are the writer's).

> ". . . it is seen at once that the size and grading of the aggregate and the quantity of cement *are no longer of any importance* except insofar as these factors influence the quantity of water required to produce a workable mix. This gives us an entirely new conception of the function of the constituent materials entering into a concrete mix and is the *most* basic principle which has been brought out in our studies of concrete."

For 42 years this assertion has survived without change although not without challenge. The following equivalent wording approximates the usual statement of what is often referred to as "the water-cement ratio law."

> "For a given cement and conventional aggregates in workable mixtures, under similar conditions of placement, curing and test, the strength of concrete is *solely* a function of the ratio of cement to the free water in the plastic mixture."

In the two foregoing formulations it is understood that Abrams (and we) are thinking and speaking only in terms of *workable mixtures* from a given cement and approved aggregates (inert, sound, compact, and adequately strong). Moreover, it is recognized that the size and grading of the aggregates are important from the standpoint of economy—the relative paste requirement for suitable workability. "Free water in the plastic mixture" alludes to the nonabsorbed water present at initial set or stiffening.

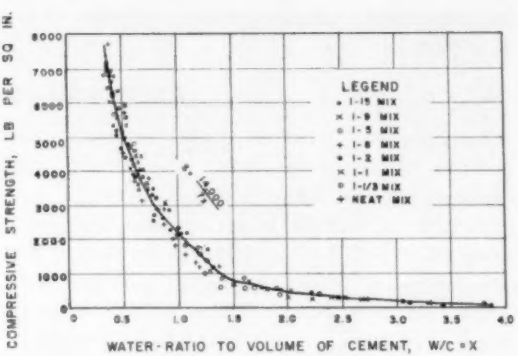

Fig. 1—Water-cement ratio versus strengths; Abrams initial curve (Reference 1, p. 3)

The two statements exclude the following as being nonrelevant variables without influence on the strength of the resulting concrete:

(a) Shape, size, stiffness, and surface texture of aggregate particles

(b) Grading of the aggregate providing it is not such as to make the mixture unworkable

(c) Amount of aggregate; the ratio of aggregate to cement (within the range of workability)

(d) Size of standard cylinder in which the concrete is tested*

Recent extensive researches by Walker and Bloem[8-10] definitely constitute a rediscovery in support of many previously published and near-forgotten allegations that the aggregate (preceding items a, b, and c) must also be recognized as on a par with the *W/C* in the extent of its influence on the strength of a concrete mixture. In other words several of the items in the following statement, (in direct conflict with the *W/C* pronouncement) seem now about to be accorded the status of proved fact:

"At equal values of water-cement ratio the strengths of workable concrete mixtures, fabricated, cured, and tested under similar conditions (in equal-size cylinders of appropriate diameter), may be influenced greatly by such factors as: The grading and relative amounts of the aggregates and by the shape, surface texture, stiffness and maximum size of aggregate particles."

The prestige and scope of the Walker-Bloem evidence establishes it as probably for many, the most significant challenge with which the longstanding *W/C* phraseology has been confronted. Convincingly it re-

*The *W/C* background tests were all on 6 x 12-in. cylinders and the generalization takes no account of the possibility that identical mixtures tested in 2 x 4-in., 3 x 6-in., 6 x 12-in. etc., cylinders might have shown different strengths per se. (This presupposes that the maximum diameter of aggregate particle for mixtures so compared would not exceed the stipulated one-fourth, or at most one-third, of the diameter of the smallest cylinder employed in the comparison). As will be pointed out subsequently[2-7] possible size-of-cylinder effect is a moot point and difficult to determine experimentally. The author is convinced that size-of-cylinder effect is (if properly isolated irom other near-inseparable effects) virtually, if not entirely, nonexistent but until this is proved or disproved size-of-cylinder effect must also be included as one of the potential unevaluated variables for which the Abram's *W/C* pronouncement makes no allowance.

focuses questioning attention on some of the presumably-settled aspects of concrete.

BACKGROUND

Initially the concept of concrete was (as it has always been for the various kinds of mortars) that of an active constituent, the cement, diluted, for the sake of economy, with an inert, inexpensive filler-type material. It was early recognized that the strength was reduced as the proportions of the diluent was increased and proportions were regularly designated in terms of the ratio of cement to aggregate. Water was an incidental constituent, to be added in whatever amount produced the desired fluidity or workability (Reference 11, pp. 575, 577).

Eventually it was recognized that as the aggregate was increased the water necessary to produce suitable workability had to be increased and that a designated ratio of water to cement (with aggregate being added by trial to produce the workability desired) gave a more sensitive and uniform basis for estimating and controlling strengths than did a specified ratio of aggregate to cement. Better control and reproducibility were attained by interchanging the longstanding roles of the aggregate and the water. The amount of the mixing water could be more easily and accurately controlled than could the amount of the aggregate. The basis of concrete control was thereby reversed, W/C becoming the measured relationship and the aggregate-cement ratio the incidental or trial factor. The W/C pronouncement was the sequel to this recognition.

Doubtless entirely unrecognized at the time was the fact that what was hailed and believed to be a revolutionary change in basic concrete technology was actually nothing more than an operational improvement; a manipulatory refinement in technique.

Elementary though the changeover actually was, it constituted a marked advance in the interest of simplification and control.

The W/C approach and the positive pronouncement based thereon, preserved in its entirety, however, the inert-filler concept of the function of the aggregate and this has been unfortunate because of the resultant conclusion that, as previously stated, so far as strength is concerned, problems of aggregate (amount, grading, surface texture, stiffness, and size) do not exist.

There can be little question that the premature writing-off of the aggregate as a potentially significant variable of concrete strength contributed substantially to the longevity of at least two fallacies in our concrete thinking:

1. Disregard of the bond between the cement paste and the aggregate particles as a vital factor in its influence on strength
2. A failure to recognize the potency of the stress-raiser effect of heterogeneity in the transmission of stress through concrete

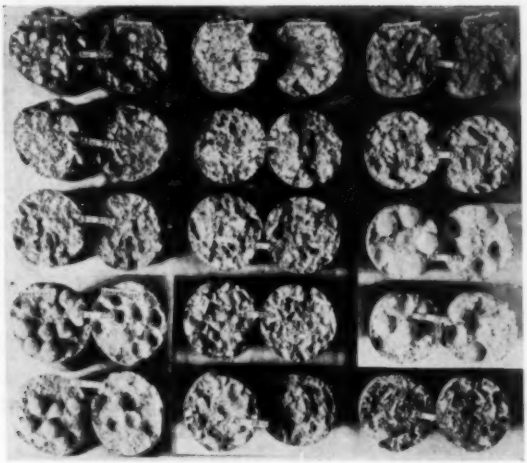

Fig. 2—Broken 6 x 12-in. cylinders; aggregate clings to upper half of cylinders as cast—water-gain effect (Reference 15, p. 243)

The initial or primary failure in concrete may occur within the aggregate, within the paste or as a slippage or severance at the junction of the two. A concrete composed of inert particles embedded in a matrix to which the particles were unbonded would obviously be much weaker than if the two are bonded.

The amount of bond developed by a given particle is proportional to the area in contact with the paste. If the particle is broken into particles averaging half the diameter of the initial particle, the superficial surface available for bond is twice as great as it was, i.e., the surface-volume ratio or specific surface, has been doubled. This is one reason why mortars are stronger than their concretes (which fact, although inconsistent with the *W/C* pronouncement, has been demonstrated repeatedly).

There is evidence that usually, perhaps always, the bond for normal concrete is the weakest of the three (aggregate, paste, and junction) and that the insipient or initial failure is a slippage or separation between particle and paste. The increasing strength of cement-aggregate mixtures as the particles become smaller (specific surface greater) has been demonstrated down to mixtures composed of small-diameter particles.[12-14]

A failure to recognize the weakening, stress-raiser effect of nonhomogeneity in a brittle material was more excusable in 1918 than it is today, for localized stress effects were less in the general consciousness then than now.

In the field of metals the importance of the stress concentrations produced by heterogeneity and nonisotropic structure has long been recognized—how the discontinuities serve as focal centers where insipient or initial (primary) failures develop. The same is true for

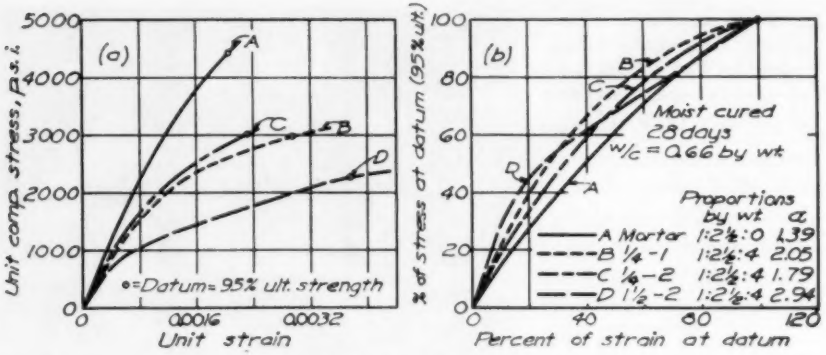

Fig. 3—A constant mortar compared with three concretes of equal coarse aggregate contents but having different coarse aggregate gradings (Reference 26, p. 31)

all brittle (and many nonbrittle) materials. Consider, for example, coarse-grained gray cast iron versus fine-grained pearlitic or sorbitic steel; the coarse texture of a granite, a conglomerate or a porphory versus the fine texture of a basalt. Even for timber, knots, shake, and grain exercise a dominant influence on the load at which failure develops.

Within any nonhomogeneous mass there are innumerable eddy currents of stress concentration from wedging, localized slippage and differential strains. Since concrete is nonisotropic we recognize broken or weakened bonds at junctions between paste and aggregate particles and horizontal planes of varying strength from water-gain effects.[15] The relative extent and importance of these factors are influenced greatly by the size, shape, grading, surface textures, and relative stiffness of the aggregate particles.

The water-gain effect is strikingly apparent in Fig. 2. The influence of the presence or absence of a coarse aggregate and of the relative maximum sizes of coarse aggregate particles (the grading) is evident in Fig. 3. Fig. 4, 5, and 6 illustrate the influence of the maximum size (and/or amount) of aggregate on the stress-strain relationship.

The foregoing figures supply from three independent approaches, (water-gain, ultimate strength, and stress-strain behavior), experimental verification that the characteristics of the aggregate are not negligible as pertinent variables in concrete.

We must keep in mind that all these variations are present for approved aggregates and plastic workable mixtures. No matter how fat and workable the plastic mixture may be the aggregate particles still occur in the hardened concrete as wedging, stress-raising discontinuities that have to transmit loads from particle to particle or from matrix to particle to matrix. The carried-over inert-filler concept of the aggre-

gate function constitutes an over-simplification that disregards several complicating realities.

As long as concrete thought continues to accept as axiomatically true the inert-filler concept, an understanding of much that needs to be recognized and learned about concrete is not possible.

EXTENUATION

Once the fallacy is recognized and conceded, the question arises: "How in a field as active as that of concrete could an erroneous concept of such magnitude survive for so long?"

This anomalous situation could have developed, and continued to exist only through a rather strange combination of circumstances, some of which may be enumerated as follows:

1. The pronouncement and the data presented in its support came from the highest of recognized sources; the evidence seemed to have covered the entire range of pertinent background mixtures; and the data presented were remarkably consistent.

2. The inert-filler concept was logical and acceptable to most; incongruous to few. It was not in conflict with the established thinking of that era for that area.

3. All the tests were on 6 x 12-in. cylinders and the maximum size of aggregate particle could not properly exceed 1.5 in. This excluded any mixtures having aggregates 2 in. and upwards. On the other hand small-aggregate concretes and mortars are usually of little practical interest and most of the initial Abrams mixtures and practically all of the millions of corroborative tests of both a control and research nature were in what might be termed the structural concrete area—graded coarse aggregates ranging from No. 4 to 1.5 in. The aggregate in both grading and amount was practically constant. *W/C* was virtually the sole variable; under these conditions the strength is a function of *W/C*. With the exception of two or three research ventures (extremely difficult to control and evaluate)[2-6] the test-

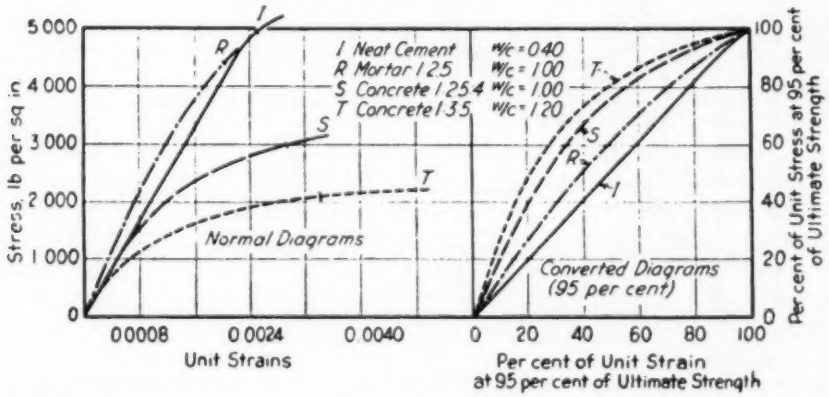

Fig. 4—Neat cement, mortar and concretes compared on basis of percentage stress-strain diagrams (Reference 25, p. 325)

ing of the types of mixtures used in dams and other massive work has been on 6 x 12-in. specimens with the oversize aggregate screened out or picked out by hand.[7] Thus the validity of the W/C concept has been assumed without any conclusive evidence for mixtures containing aggregate larger than that common to the structural range of practice.

4. Trying to secure mixtures of uniform workability and repeatable proportions and strengths had proved to be almost impossible and concrete practice was in dire need of a dependable simplified technique of design. The water-cement ratio with its apparent validity and great simplicity was most welcome and the concrete world was conditioned for its ready acceptance and adoption.

5. Of the greatest practical importance of all, possibly, was the fact that the organization under whose sponsorship this startling innovation was achieved, was also vitally interested and was able to promote the theory and its applications with virtually unlimited resources and talent.

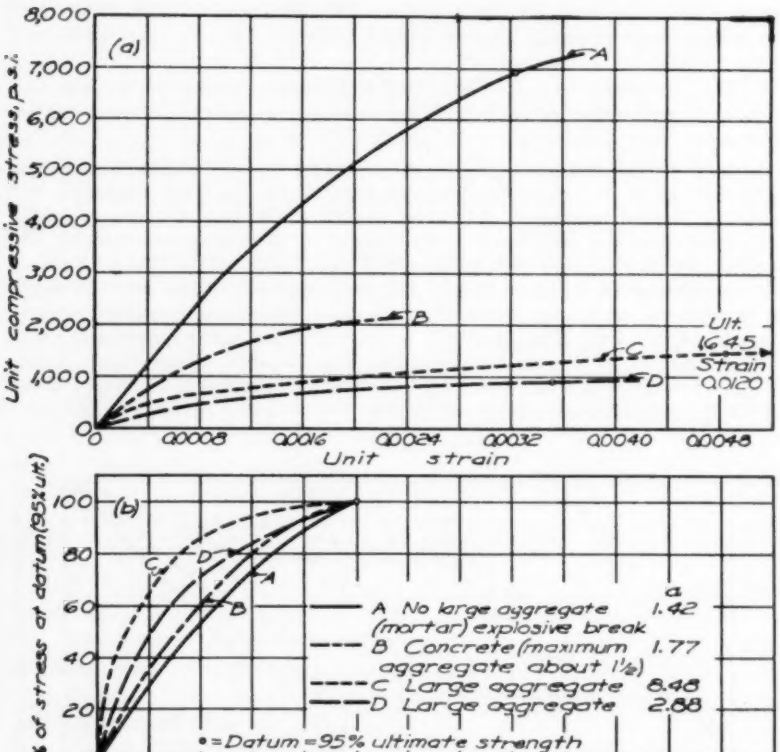

Fig. 5—Drilled cores from a dam compared on basis of percentage stress-strain diagrams; mortar, nominal concrete (about 1.5 in. maximum aggregate), and large-aggregate concrete (Curves C and D contained portions of plum inclusions) (Reference 26, p. 32)

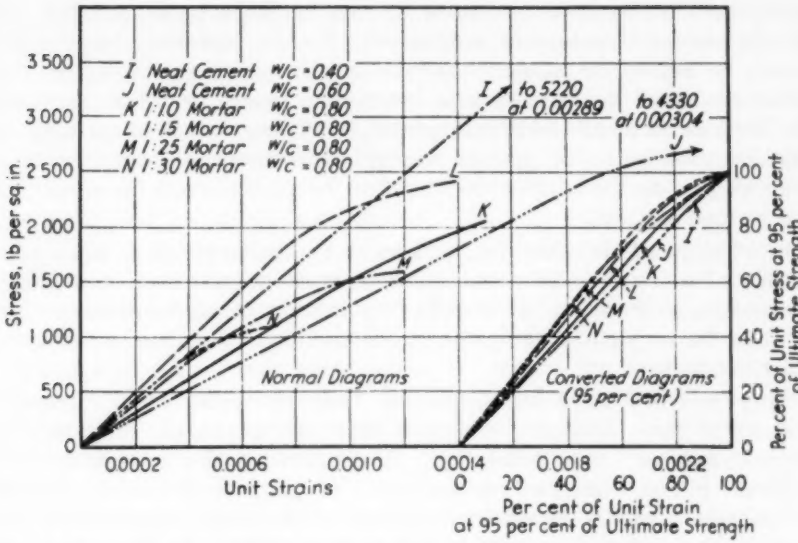

Fig. 6—Comparisons of neat cement mortars with sand-cement mortars that vary from rich to lean on basis of percentage stress-strain diagrams (Reference 25, p. 322)

With such an accumulation of favorable factors, including great masses of seemingly valid experimental verification, it is no wonder that the concept "took" and that practically all of the leading workers and authorities in the field of concrete were convinced that here, in one simple pronouncement, was the cure for much of what had been troublesome about concrete.

True, there were from the beginning the inevitable few disbelievers—the agnostics. Invariably these were from among the fringe of persons who in the course of their researches or experience had by intent or inadvertence tapped other areas of aggregate size, grading, etc., and whose findings were not in agreement with those so consistently shown in Fig. 1.

Some of these discordant data were from individuals or laboratories not heretofore well known in concrete and since concrete experimentation is exacting and rather involved, the apparent queerness of results could be readily explained or discounted as being probably due to unworkability, leakage from the mold during the interval between placement and initial set, or absorption of moisture by the aggregate. There was usually a presumption that the cause was due to oversight or lack of control stemming from inexperience or ineptitude.

Since the earliest years of the *W/C* era the author has been a spectator and participant as one convinced that the generalization is too

sweeping. His contributions have at various times been accorded each of the several treatments mentioned. Never, however, has he had reason to believe or suspect that the counter contentions of the *W/C* adherents were inspired by any intentional desire or effort to distort the facts or to shade their interpretation. There has been no question that the believers are just as sincerely convinced of the validity of their dogma as the author continues to be of the need for qualifying the pronouncement.

In other words it is but fair and correct to reiterate that this alleged fallacy of more than 40 years standing must be accepted as a sincere conscientious sequel to a strange combination of background circumstances, the major one being the unquestioning acceptance of the inert-filler concept.

Only recently have the extensive tests of Walker and Bloem,[8-10] seemed to have produced evidence that can no longer be talked or rationalized out of consideration. Illustrative of the completeness of a library burial, these two recognized investigators had initially thought of the recent totally unexpected indications as a *new* discovery rather than as the *re*discovery which, subsequently, they were not hesitant to recognize and admit it to be.

In spite of its relative paucity, the literature of dissent is itself of sizable volume and the forthcoming review necessarily is spotty, sketchy, and assertive. Excerpts are cited only for items that are deemed especially pertinent and which might in their initial publications, be somewhat less available than, for example, material of the ACI or ASTM proceedings. The cross-references and lists of literature cited in the various items should suffice to reopen the subject more completely for one desirous of rounding out the coverage.*

BACKGROUND OF THE AUTHOR'S DISSENT

The author's first active contact with the problem came at the University of Illinois in 1923 as the by-product of a graduate thesis[16] under the immediate direction of Talbot and Richart. The project was conducted jointly with the engineering experiment station, being an investigation of 31 coarse aggregates. About ten of these were conventional crushed stones, crushed slags and gravels; several were semiconventional including Haydite and cinders. Others were crushed bricks of varying hardness and absorptivity, including hardburned tile. About ten of the aggregates were freaks, including coke, cork, wood blocks, charcoal, broken-up stove iron, iron balls (1.50 and 1.87 in. in diameter), fence staples, fragments of ¼ in. plate glass, and a thinner Florentine

*It is with apologetic self consciousness that the author's name is so often drawn into the reference picture. This is not to imply that he considers his contributions to have been more significant than those of others of the dissenters. He has, however, lived with the subject more intensively than have most. Moreover one's own efforts are inevitably more in the immediate consciousness.

glass. The objective was to secure evidence on what might be expected when aggregates that violated the generally-recognized characteristics of acceptable aggregates (chemically inert, reasonably free of flat or elongated particles, nominal surface textures, and at least as strong as the mortar) were used, all in the same solid-volume amounts (0.40), in the same cement-sand-water mortar matrix. Grading varied widely but all fragments except the 1.87 in. diameter iron balls lay between the No. 4 and 1.5 in. sieve sizes.

Because of the great range in rate and amount of absorptivities, the 6 x 12-in. specimens were cast with the coarse aggregates in three different states: dry, saturated, and after 15 min of soaking, equal solid (absolute) volume proportions being used throughout. The significant (comparable) specimens were those in which the coarse aggregate entered the mixture, saturated and drained (approximating the condition recognized currently as saturated surface-dry). Specimens were fabricated singly, each one-cylinder batch being weighed out and mixed separately. Tests were in the usual 28-day standard-cured condition and stress-strain data were secured semiautographically from each specimen without interrupting the continuous application of load.

Since all specimens were to have an identical mortar, (sand-cement ratio 3.2 solid volume—1:2.64 by weight—1:2.22 bulk volume) and a *W/C* of 0.71 (5.3 gal. per sack), it seemed logical that the constant mortar should be made to serve as the norm with which the strengths of all the concretes would be compared. At intervals, therefore, there was mixed one batch of the mortar from which were cast one 6 x 12-in. and two 2 x 4-in. specimens—stored identically and tested at the same 28-day age. The project included 361 concrete specimens, 113 6 x 12-in. mortar specimens and 224 of the 2 x 4 in. companion mortar specimens.

In addition to the foregoing, two complete water-cement ratio versus strength series of the 6 x 12 in. mortar specimens were cast, ranging from the unworkably dry to the overly wet mixtures.

Much to the surprise of all, the mortar specimens consistently gave strengths that were from 15 to 25 percent in excess of those from the orthodox-aggregate concrete specimens containing the same mortar. Moreover, the *W/C* versus strength curves were correspondingly higher than the locus for the normal concretes having the identical *W/C*.

Obviously this was a jolt and during the next several years this inadvertent discovery that the strength of a mortar is in excess of that of its concrete was a focal point for attention in connection with much reconnaissance testing by the author at the University of Colorado. The findings were consistently the same.

Eventually in 1927 many of these findings were recorded in the author's second paper before the ACI.[17] As flagrant heresay this attracted attention, gaining some support; and also drawing fire. Much

of the ensuing interchange is as timely today as it was 34 years ago. For anyone interested in reviewing *W/C* thought and counter-thought this 1927 paper and the accompanying pro and con discussion are commended.

OTHER EARLY OBSERVATIONS OF RECORD

Hutchinson[18] as one of his conclusions (2a, p. 155) states: "The compressive strength and the resistance to impact of concrete are increased as the size of coarse aggregate is decreased . . . ." Walker, in discussing the paper (p. 177) applied the fineness-modulus criterion and rationalizes on the basis of probable coarseness with resulting weakness from nonworkability to which the author replied that there was no detectable lack of workability. He added that the results secured from drilled cores from many miles of pavement indicated decisively that whenever, from incidental segregation, the aggregate in the core was on the small side, the strength was noticeably and consistently higher. In the same discussion, Richart (p. 177) mentions "recent results" (comparable to those of the modern Walker-Bloem[8-10] rediscoveries), in which aggregate graded No. 4 to ¾ in. unexpectedly gave *high* strength and that from ¾ to 2 in. unexpectedly gave *low* strength. Slater (p. 178) reported a similar finding which proved "puzzling" until *rationalization* was accomplished via fineness modulus, *et al.*

From supplementary researches while serving as concrete technician on the Bull Run Dam near Portland, Ore., Powers[19] became convinced that *W/C* versus strength is only part of the story. Lyse, in discussion of the paper, succeeded in getting all the straying points of Fig. 1, p. 422, back on the curve "where they belonged." There is too much about these tests that is not known for certain to attempt a final appraisal but as indicated in the closure, Powers seemed convinced that his subsequent experimentation reinforced the initial indications.

Reference 12 provides evidence of extreme fluctuations within the "water-cement ratio law." Note especially the great range of strength ratios (Table 2 Column 6, p. 627), and the summary (pp. 633-634). As evidence of how far the *W/C* can come from being applicable, Reference 12 falls in the same category as References 13 and 14.

Collier[20] directs attention to practical inconsistencies in the *W/C* relationship. The tests were not under complete control and certain weaknesses are brought out in the discussion. In the Gilkey discussion (Table 1, p. 747) it is noted that the data from McMillan[21] do not agree with Abrams (Reference 1, Fig. 1), that neat cement paste fits the *W/C* versus strength relationship right along with all the others of the eight mixtures shown in Fig. 1. That the *W/C* versus strength curve for neat cement mixtures lies above those for sand-cement and for concrete is also in accord with the author's observations (Fig. 7).

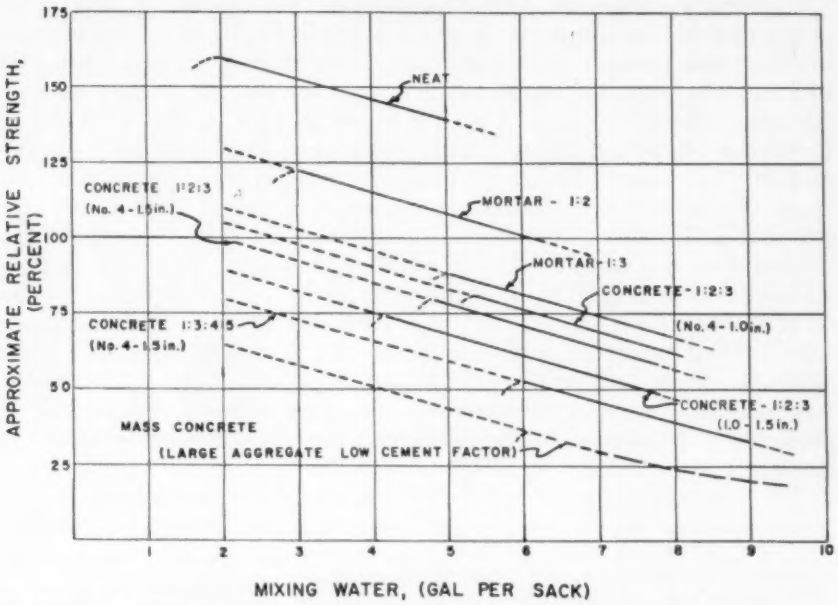

Fig. 7—Water-cement ratio curves; approximate extreme workable ranges for a few mixtures of different proportions and/or coarse aggregate gradings

The strength of a fairly rich mortar (say 1:2) is about 1.3 times the strength of its normal concrete (say 1:2:3.5) and the strength of a neat cement paste is about 50 percent greater than that of the corresponding concrete (assuming that the W/C curve for concrete could be extrapolated to produce a workable comparable mixture). In other words W/C versus strength curves for mortars and neat cements can be expected to lie about 30 and 50 percent higher, respectively, than does the W/C curve for an average structural concrete.

For the Abrams' plotted data (Reference 1, Fig. 1) of the mortars and neat pastes all line up with the concretes on the one basic curve. On that curve the mean strength for the neat cement paste was about 7500 psi at a W/C of about 0.40 or 3.0 gal. per sack. In compiling laboratory data for the set of curves which McMillan[21] shows as Fig. 4, the fact that pastes overshot the equation $S = 14{,}000/7x$ was evidently recognized and at a W/C of 3 gal. per sack the strength is shown as 10,000 psi, about 50 percent above where the Abrams' equation would have placed it.*

*Fig. 4 is itself queer however. The nonconforming lefthand portions of the curves descend smoothly to the right joining the conventional Abrams curve at about 5 gal. per sack. Strangely, Reference 21, Fig. 4, shows actual plotted points presumably indicating that these curves were the result of specific experimentation. Note again that Fig. 4 is for cement-water pastes that extend to 7.5 gal. per sack. This presents an incongruous situation since even at less than 5 gal. per sack, a neat cement paste, takes on a consistency comparable to that of cream. While Fig. 4 does definitely correct a discrepancy in the Abrams relationship, a non-experimental generalization seems to have been misplotted and somewhat overextended.

A primary conclusion from the evidence resurrected in this paper is that the W/C relationship is really a whole family of relationships—more or less parallel to one another. Each type of mixture not only has its own series of water-cement ratio curves but it has its own limited workability range. Fig. 7 is based in part on the mixtures of Reference 14, being simply a generalized but representative approximation. It illustrates reasonable values for relative strengths of different types of mixtures and indicates the extreme ranges of workability (too dry and stiff at the left; too fluid at the right). It also illustrates the limited area embraced by the structural type of mixture which supplies virtually the sum total of background evidence on the Abrams' version of W/C versus strength. Percentage comparisons of relative strengths may be best noted along the ordinate that corresponds to the driest workable cement-water mixture (about 2 gal. per sack). The leaner, larger-aggregate types of concrete—the mass concrete area— would presumably fall below in the general vicinity of the bottom (dashed) curve.

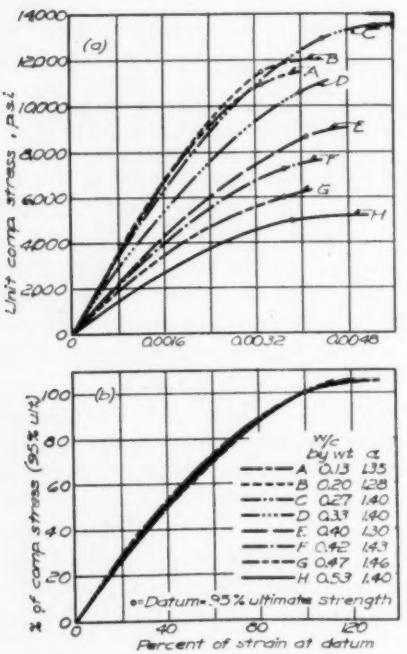

Fig. 8 — Water-cement ratio versus strength compared on basis of percentage stress-strain diagrams (lower); neat cement, 2 x 2 x 4-in. prisms at 28 days (Reference 26, p. 27)

Weymouth[22] placed great emphasis on a rather elaborate and highly conservative grading technique. Gradings meeting the Weymouth criteria were invariably oversanded as compared with our usually-acceptable mixtures. Powers, in discussing this paper (p. 378), writes: "It should be noted that Mr. Weymouth's work does not deal with the relation between water ratio or voids ratio and strength; it deals with factors of mix composition which control the water or voids ratio. *What the strength at a given water ratio may be depends on many other factors*" (Italics are the author's).

In the author's discussion of the Weymouth paper[22] there are also quotations of test results from Thomas and Graf[23] and from Gause[24] all supplying additional evidence on strength variations "within the law." The other discussions, contributed as they are by recognized leaders in the field, cover a variety of aspects and bring

out many points that bear heavily on effects of grading and also on some of the reasons for differences in strength, permeability, and other properties at a constant *W/C*.

In Abrams' discussion (Reference 22, p. 384) is a statement ". . . so long as we use workable mixes and *unless we go too far afield in aggregate grading* we find that strength is a function of water-cement ratio . . ." (Italics are the author's).

Here, for the first (and only) time within the author's awareness Abrams states that the "relationship" has limitations within the "law." "Too far afield" is admittedly vague and general but it is definitely a concession that opens the way for the *W/C* relationship to assume its correct role as *one of several factors* that exercise an important influence on the strength of a workable portland cement concrete, paste, or mortar.

This qualification, entered the literature more than 22 years ago. Yet it has been so thoroughly buried that Walker,[8,9] was surprised by the 1956 revelation that the grading of the aggregate can (and does) exercise a significant influence on strength.

OTHER EVIDENCE

The *W/C* versus strength aspect is a by-product of the ASTM Committee C1's subcommittee report of 1938.[13,14] But for the author's recognition of its great significance (as regards *W/C*), its contribution to the evidence on size-of-aggregate effect (extending even into the small-particle fine-aggregate range) would probably have remained unnoticed. Since the *ASTM Bulletin* is likely to be less readily available than the *Proceedings* of ASTM, this reference is reviewed in some detail.

In an ASTM Committee C1 search for a standard mortar that could serve as a basis for comparing the concrete-making characteristics of different portland cements, 14 laboratories conducted identical test programs on three mixtures. They were all mortars but two of them, the B and P mixtures resembled "baby" concretes. Both had fine aggregates (Ottawa banding sand and finely ground Potter's flint, respectively) and ¼ to ⅜ in. pebbles for the "coarse" aggregates. The third mixture (C 109, evolved some years previously) used a specially graded sand (fineness modulus 1.72) and none of the "pebble" coarse aggregate. All three mixtures had identical water-cement ratios of 0.80 and the best of plasticity. Quotations are from the initial subcommittee report[13] and from the discussion.[14]

From the committee report (Reference 13, p. 269) . . . "the strengths of the pebble mortars are decidedly higher than the strengths of the C mortar, the average of the pebble mortar strengths being 62 percent higher than those of the C mortar at 1 day, 53 percent at 3 days, 52

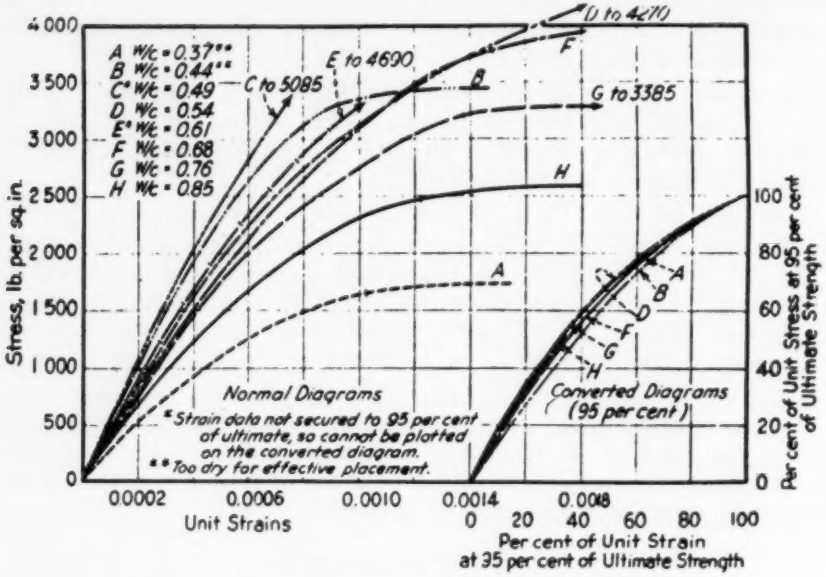

Fig. 9—Water-cement ratio versus strength compared on basis of percentage stress-strain diagrams (at right); mortar 1:2.6 by weight, 6 x 12-in. cylinders; age 28 days (Reference 25, p. 321)

percent at 7 days, 47 percent at 28 days, and 53.5 percent average for the four periods. These results are quite unusual since the cement-water ratios used were identical. It is not the purpose of this investigation to attempt to explain these differences in strength. . . . In this connection laboratory No. 4 made tests using materials similar to these, the same cement-water ratio and a continuous grading from No. 100 to 3/8 in. (five fractions) in accordance with Weymouth's theory and obtained strengths of the same order . . ." All data were remarkably consistent. The compressive strengths of the weak mortar (C mortar) were much nearer those of a comparable concrete (same W/C) than were those of the "baby concretes" (pebble mortars) in which the great excess over the predicted concrete strength was so pronounced.

Let us now refer to the discussion (Reference 14, p. 33). In the preliminary interchange between the author and committee C1 chairman, P. H. Bates,* Bates writes:

> ". . . When I first proposed some time ago to Mr. Moore,† mortar of the type referred to in C1's report together with preliminary data, I was wondering whether I would not be told it must all be wrong, because our

*Long-time Chief, Silicate Clay Products Division, National Bureau of Standards.
†Then director of research, Universal Atlas Portland Cement Co., and chairman of the subcommittee in charge of the tests.

'hallowed' cement-water ratio did not hold. But time has passed and you now 'bob up' as the first one to point out with any positiveness the fact that the mortars in question do unquestionably show the failure of the water-cement ratio as a generalization . . ."

Gilkey to Bates . . . "These results are 'unusual' to the members of the committee only because they were precocious in learning and steadfast in believing that the water-cement ratio versus strength relationship was, and is, the alpha and omega of concrete. Having, in common with virtually the entire concrete world, accepted as final this useful but grossly overstated (though oft-reiterated) generalization, it is not surprising that such contrary evidence, when finally thrust upon them, should be rated queer.

"It so happens that the writer has, for the past 15 years, been afflicted with a morbid curiosity which has led him into many ventures sufficiently academic and impractical to supply him as a steady diet, with results which were unorthodox to most, unbelieved by many, and of little interest to others. Among these results are some likely to be quite disturbing to the souls of pious *W/C* adherents. The representative data of the accompanying

TABLE 1 — COMPARISONS OF RICH AND LEAN CONCRETES AND THEIR MORTARS AT VARIOUS AGES, CURINGS, AND TEST CONDITIONS; WATER-CEMENT RATIOS THE SAME FOR ALL*

Mixture by weight	Coarse aggregate size	W/C, net	Slump, in.	Compressive strength, lb per sq in.							Average, percent
				Standard cured				Variable curing,† age: 1.5 years			
				7 days	28 days	3 months	1.5 years	SD	AD	AW	
1:2:0	—	0.86	11.0	2250 (100)	3060 (100)	3900 (100)	5130 (100)	6170 (100)	3030 (100)	2020 (100)	(100)
1:2:3	1-1½	0.86	7.6	1320 (59)	1980 (65)	2520 (65)	3160 (62)	3460 (56)	1920 (64)	1600 (79)	(64)
1:2:3	¼-1½	0.86	7.8	1650 (73)	2400 (78)	3170 (82)	3860 (75)	4160 (68)	2170 (72)	1510 (75)	(75)
1:3:0	—	0.86	1.9	1780 (79)	2680 (88)	3440 (88)	3740 (73)	4350 (71)	2510 (83)	1890 (94)	(82)
1:3:4½	1-1½	0.86	0.0	1270 (56)	2060 (67)	2440 (63)	2380 (46)	3160 (51)	1850 (61)	1380 (68)	(59)

*From Reference 14, p. 35. Fine aggregate was a washed natural sand and coarse aggregate was crushed quartzite. All specimens, 6 x 12-in. cylinders.

†SD = Standard cured for about 13 months and exposed to dry laboratory air for about 5 months prior to testing dry. One month air exposure should have produced same results.
AD = No moist storage. In the dry air of the laboratory after removal from mold at age of 1 day.
AW = Same as above but saturated by 24-hr immersion just prior to test.
Some of the indications of Table 1 may be summarized as follows:

1. The 1:2:3 concretes have 64 and 75 percent of the strengths of their 1:2:0 mortar.

2. The 1:3:4½ concrete has 72 percent of the strength of its 1:3:0 mortar.

3. The 1:3:0 mortar has but 82 percent of the strength of the 1:2:0 mortar, and is only slightly stronger than the 1:5 concretes.

4. At constant water-cement ratio, strength is reduced as the aggregate ratio is increased, but the reduction is more pronounced for increases in sand ratio than in coarse aggregate ratio, although the effect of changes in neither is negligible.

5. Here the graded coarse aggregate (¼-1½) produces stronger concrete than does the one-size (1-1½) coarse aggregate. There is nothing in these data to indicate whether the difference is due to the grading or to the difference in the average size of particle. Other tests show that small sized particles reduce strengths less than an equal weight of larger ones.

6. Strength ratios are reasonably constant for mixture comparisons at different ages, curings, and test conditions.

Table 1 are not in all respects parallel to those secured by the committee, but they should serve as a basis for demonstrating the reasonableness of the committees' findings."

Within conventional sizes of aggregates the mixtures of Table 1 illustrate both size and amount-of-aggregate effects paralleling some of the significant differences present in miniature in the committee's B, P, and C mixtures. (B and P are miniature concretes with rich mortars of finely graded material with pebbles for coarse aggregate. Let us recall that the C mixture is a moderately lean* graded mortar with nothing that can be considered as approximating the coarse aggregate in concrete.)

In Table 1, Mixtures 3 and 4 supply a comparison between a fairly rich concrete and a fairly lean mortar. In this case the strength of the 1:5 concrete does not overtop that of the 1:3 mortar but if the mortar were a trifle leaner it undoubtedly would. Comparisons of concretes with their mortars are afforded within the first three mixtures and the last two.

Table 1, although in itself representing a limited reconnaissance series, does indicate relationships and trends some of which are eloquently supported by the Committee's published results and others, of which have been fully verified by other series and other investigators. All such data show that within the W/C relationship (at constant W/C):

1. The strength of the concrete is not the strength of its mortar
2. Decreasing the average size of the aggregate increases the strength†
3. Increasing the amount of the aggregate decreases the strength
4. Fine aggregate has a greater influence on strength than does an equal amount of coarse aggregate

SIZE OF CYLINDER

As previously noted, the W/C generalization not only disregarded size, grading, and amount of aggregate for workable mixtures, except insofar as these might bear on the wetness of the mixture required for workability, but it failed of any qualification for possible size-of-cylinder effect. As large aggregate, up to 6 in. (and in some instances to 9 in. or more) became routine for massive structures such as dams, important questions arose.

To test large-aggregate concrete in cylinders appropriate to the size of the aggregate (diameter of cylinder, at least four times the diameter of the largest aggregate fragments) left two potential unevaluated

*It is, of course, the leanness of the C109 mortar that not only provides it with a suitable consistency but also with a strength that is comparable with that of a concrete having the same W/C.

†There must be a lower limit to the applicability for Item 2 but it seemed not to have been reached with the small particles of the committee's mixtures B and P.

variables. Obviously the only way actually to determine whether or not either size (and grading) of aggregate, or size of cylinder, or both, introduced significant additional variables called for:

1. The large scale testing of standard cylinders covering the complete range of size that might be used, with the same mixture in all sizes of cylinder. If the smallest cylinders were 2 x 4 in. (only suitable for aggregate 0.5 in. or smaller) that mixture would have to be used for this entire series—1.5 in. aggregate concrete if 6 x 12 in. was the minimum size of specimen.

2. Then an independent size-of-aggregate series could be run in which the largest size of aggregate appropriate to each size of cylinder could be employed.

Of course identical water-cement and sand-cement ratios would be used throughout both series but even with that, adequate, properly controlled data would be difficult to secure.

Recognized size-of-cylinder variables include: (a) possible differences in temperature due to undissipated heat-of-hydration effects on the interior of a large specimen, (b) different type of response to load for material surrounded by a considerable mass of concrete, (c) differential volume change effects—shrinkage and/or swelling.

Unrecognized but probably more disturbing are such practical aspects as: (a) the fact that in general different types and capacities of testing machines must be used for different sizes of specimens, involving such differences as rate of testing-head travel and rate of unit strain, (b) the all-important end-condition effects— ground to a plane, capped with sulfur-sand mixture, neat cement, or plaster of Paris, (c) comparable uniformity of moisture and curing conditions throughout the small and large masses.

Size-of-aggregate effects would have to be evaluated after the data secured were already "corrected" for whatever had been determined as the *presumed size-of-cylinder effect*. Without exhaustive testing programs, the best of control, and statistical processing of data, conclusions from such a two-step operation would be speculative at best.

Within the 2 x 4 in., 3 x 6 in. and 6 x 12 in. cylinder range the author has found no consistent or appreciable size-of-cylinder effect. Some report slightly higher strengths for the smaller sizes of standard cylinders but in view of the inherent uncertainties and the small range of differences reported, no size-of-cylinder effect within this size range can be accepted as an established fact.

There have been three projects designed to probe size-of-cylinder and/or size-of-aggregate effects within the large-cylinder range. These are reported in References 2 and 3, 4, and 5 and 6. These three projects all entailed uncertainties sufficient to make any conclusions tentative but the weight of the evidence seems to be against any significant

size-of-cylinder effect but supporting the pronounced size-of-aggregate effect already demonstrated in the small size ranges.

From the brief initial National Bureau of Standards report on the Santeetlah Dam specimens[2] one tends to gain the impression that there is a significant size-of-cylinder effect. From the subsequent careful attempt to analyze the data,[3] however, it appears that the differences in strength are not believed to be, in any degree, size-of-cylinder effects—to quote ". . . it was found that for cylinders of different size and made of the same proportions, and by the same fabricating and curing methods, the strength was practically the same, and conversely every change in strength was accompanied by a change in fabricating method or a change in mix. . . . The difference in cylinder size was therefore shown not to be responsible for the change in strength."

This leaves size-of-aggregate as the only significant variable (all were at the same W/C), which is in accord with much of the evidence that has been cited.

For the Sturm project[4] the mortar tests from 6 x 12-in. and 8 x 16-in. cylinders tested 30 percent stronger than the concrete tests from 24 x 48-in. cylinders. Here we have both a size-of-cylinder and a size-of aggregate variable. Large cylinders of *concrete* were strikingly weaker than the smaller cylinders of identical *mortar* but the dual variable leaves an opening for interpretational differences.

Blanks and McNamara reported important size-of-cylinder effects and no significant size-of-aggregate effects (Reference 5, pp. 300-301, Conclusions 1 and 4). The author places exactly the opposite interpretation on the evidence presented (Reference 6, p. 235-248). This leaves the Blanks-McNamara results negative, positive, or indeterminate dependent on how the reader appraises the controversial evidence presented.

It does seem certain, however: that concrete containing large aggregate, tested in large cylinders will have a substantially lower strength than will concrete or mortar (of the same W/C but smaller aggregate) tested in smaller cylinders. This by-passes as possibly controversial and in some degree speculative, the reason why the large cylinder, large-aggregate concrete is weaker. Nor is there an adequate basis for estimating the extent of the differential for a particular combination of aggregate and cylinder size.

THE McMILLAN SUGGESTION

As a method to measure the strength of large-aggregate concrete McMillan[7] offered the suggestion of wet screening (or picking out the oversize fragments of aggregate). If the remaining mixture tended to be too fluid, aggregate of smaller grading could then be added to

"thicken" the mixture. Removing the large aggregate has allegedly strengthened the resulting small-aggregate specimen. Adding aggregate to stiffen the mixture has weakened it. How much is wholly indeterminate. In the 13-page discussion (Reference 7, p. 522-535) triggered by the innocent 1-page suggestion (Reference 7, p. 521) the author touches on quite a range of matters basic to concrete and its problems.

EVIDENCE FROM STRESS-STRAIN BEHAVIOR

The percentage stress-strain diagram[25,26] constitutes a useful device for comparing the basic response of materials to load. Fig. 8 shows a *W/C* series (*W/C* the only variable) for 2 x 4 in. neat cement specimens and Fig. 9 shows a similar series for 1:2.6, 6 x 12 in. 28-day sand-cement mortar specimens.

Both series reduced to the percentage basis show that in spite of the great range in actual strengths, the differences in basic or qualitative behavior are almost indistinguishable. The same basic similarities exist for other important variables such as age at test, condition at test (saturated or airdried, etc).

On the other hand Fig. 3, 4, 5, and 6 illustrate the pronounced effect of the nonhomogeneity incidental to variations in maximum size and amount of aggregate. The same kinds of nonhomogeneity effects were strikingly apparent in plaster of Paris paste in which both lead shot of several sizes, and standard Ottawa sand were used as aggregates (Reference 26, pp. 33-34).

Incidentally, when lateral strain data are secured as is done if Poisson's ratio is to be evaluated (Reference 26, Fig. 6), the lateral stress-strain diagram is likely to be an even more sensitive indicator of the internal disturbance due to nonhomogeneity.

It appears that a primary or incipient failure of a heterogeneous specimen under load is usually, if not always, a slippage between matrix and aggregate particles; more pronounced and starting at lower loads as the aggregate is increased in size or amount. Such a concept seems to be fully in accord with the evidence from stress-strain behavior as well as from the strength tests.

EVIDENCE FROM WATER GAIN

There is still another evidence of the potency of nonhomogeneity in its effects on the behavior of portland cement mixtures under load. The well-known water-gain phenomenon[15] results in the accumulation of free water and air bubbles on the lower surfaces of all aggregate particles. The larger the fragment the greater is the accumulation over a continuous surface. By the same action—essentially a sedimentation—the upper surfaces lose some of their accumulations.

The bond between aggregate and matrix is improved slightly at upper contact surfaces and is weakened at the lower ones. This results in unequal adhesions and resistances to slippage or separation at different contact surfaces. These add to the effects of nonhomogeneity incidental to the size, amount, and grading of aggregate.

Fig. 2 illustrates the pronounced clinging of the aggregate particles to the material that was directly above it during the plastic phase. The specimens were broken by dropping on to a mandrel located at midlength. Only an unusually well-embedded fragment clings to the lower half of the specimen as cast.

CONCLUDING COMMENT

It may be observed that little of the evidence presented is in actual conflict with the recorded background evidence of the *W/C* pronouncement. The differences are almost entirely ones of interpretation and of the scope accorded to the purported verification.

There is entire agreement that the sweeping Abrams pronouncement is well supported in the area of structural concrete (mixtures from graded aggregates not exceeding 1.5 in. maximum size). The negligible coverage in the neat-cement, mortar, or small-aggregate area missed entirely, however, such striking indications as those of the ASTM Committee C1 tests.[12-14] Nor was cognizance taken of the complete lack of information at the time of the pronouncement, relative to the possibility of contrary indications in the large-aggregate, and/or large-specimen range.[2-7] Apparently, the faith in the inert-filler concept was so implicit that it obviated the sense of need for a thorough experimental verification for either small-aggregate or large-aggregate mixtures.

Had the interpretations of the evidence and the formulation of the *W/C* pronouncement been so qualified as to restrict them to the area that had actually been explored adequately, the pronouncement would have been validly useful, although admittedly it would, so qualified, have lost somewhat in glamour and rhetorical appeal.

SUMMARY

1. In the light of such evidence as that herein discussed or cited the author believes that the *W/C* pronouncement, as initially formulated and as it still stands, constitutes a serious oversimplification that blocks sound concrete thinking.

2. The alleged fallacy apparently stems from an unquestioning acceptance of the long-standing concept of the aggregate function as being solely that of inert filler, disregarding such factors as the heterogeneity and nonisotropicity of concrete.

3. The essential validity of the *W/C* pronouncement has been repeatedly verified for mixtures that might be designated as within the structural concrete range (graded aggregate up to 1.5 in. maximum size, tested in standard 6 x 12-in. cylinders).

4. For unusual gradings in which the maximum sizes and/or amounts of the aggregate depart substantially from the "structural" type of mixture:

(a) The backlog of experimental exploration was insufficient to justify the wording of the pronouncement

(b) On the basis of a substantial accumulation of evidence from a variety of sources, the pronouncement as made is not valid and can be seriously in error

5. The wording of the *W/C* versus strength pronouncement should be revised to read substantially as follows:

"For a given cement and acceptable aggregate, the strength that may be developed by a workable mixture of cement, aggregate, and water (under the same mixing, curing, and testing conditions) is influenced by:

(a) The ratio of cement to mixing water

(b) The ratio of cement to aggregate

(c) The grading, surface texture, and stiffness of the aggregate

(d) The maximum size of the aggregate

Where more than one of the foregoing are appreciably present as simultaneous variables, each of the variable factors will exercise a proportionate influence on the strength of the concrete."

6. For workable mixtures that lie within one general range of size, grading and amount of aggregate the major strength variation will be contingent on the ratio of cement to water. Within any one such limited range the appropriate *W/C* versus strength relationship constitutes a valauble tool in the proportioning and regulation of mixtures. In generalized form the *W/C* versus strength relationship can be represented by a family of overlapping, approximately parallel curves similar to those shown in Fig. 7.

7. Size-of-standard-specimen per se remains unevaluated with regard to the possible influence on strength. While limited tests supply indications that, within the range of 2 x 4 in. to 36 x 72 in. specimens, size-of-specimen in itself is not a major variable, the subject is controversial. Valid appraisals are especially elusive because of inherent difficulties in keeping comparably under control such practical aspects as the ends of the specimens, rates of loading, types of testing equipment, moisture, and volume change, heat-of-hydration effects, *et al.*

8. Stress-strain relationship and the water-gain phenomenon are cited as visual evidence of the effects of the heterogeneity of any such material as concrete.

9. It has been the mission of this paper to reinstate past dissents and to reinforce recent *rediscoveries* by attempting to assemble in one place references to, and brief discussions of, some of the more significant of the earlier contributions; also to supply supplementary evidence of various sorts, all blending to indicate the long overdue need for a bigger concept and broader thinking relative to the functions and limitations of the aggregate as a constituent of concrete.

10. Nothing herein tends to deny or belittle the dominant role of the water-cement-ratio as *the* design criterion for mixtures from similar materials that lie approximately within one general range of proportions, size, and grading. Over any one area (as, for example, that of structural-type mixtures) the *W/C*, trial-batch technique maintains its status as unexcelled for simplicity and dependability. For important work the use of the "job curve," long recommended by the Portland Cement Association, continues to provide a workable and highly satisfactory design procedure. Recognition and acceptance of the qualifying factors herein discussed and emphasized will contribute substantially to the broadening of concrete thinking without any significant alteration in current design techniques over the usual range of concrete practice, while at the same time the informed practitioner stands alerted for taking design account of significant differences heretofore disregarded on the assumption that their influence on strength was nil.

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Title No. 57-56

Precast and Prestressed Folded Plate Slabs

By HARRY H. EDWARDS

Prestressed precast folded plate concrete slabs offer tremendous potential for widespread use in buildings for roof, floor, and wall construction and in highway bridges.

A relatively low capital investment in prestressing facilities will produce a complete span range of folded plate products limited only by the ability to deliver and erect. The cost of folded plate structures promises to be competitive with other building materials for long spans and heavy loads.

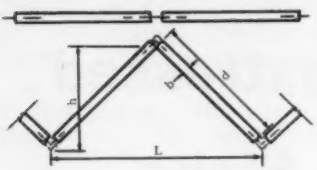
Prestressed folded plate offers the opportunity of bringing the advantages of concrete construction—strength, fire resistance, blast resistance, and low maintenance cost—into buildings of all price ranges, and in addition, makes it economically feasible to design concrete structures in the 50- to 120-ft span range.

■ A PRESTRESSED CONCRETE FOLDED PLATE, constructed of two 2½ in. flat slabs, 51 in. wide and 50 ft long, joined at the ridge by welding together protruding #3 bars at 16 in. on centers was test loaded by the author's firm. The cross-sectional dimensions of each folded plate were 72 in. x 37.75 in. Six 7/16 in. strands were used to prestress the member. Wire mesh was used as temperature reinforcement. The joint was grouted.

Using the simple beam theory for design, the folded plate was designed for a safe superimposed load of 30 lb per sq ft. The maximum amount of tensile stress allowed was 250 psi.

The folded plate was test loaded until an equivalent symmetrical loading of 158 lb per sq ft was reached; more than five times its design live load. After losses, the fiber stresses at the supports due to prestress were calculated to be 231 psi tension in the top and 943 compression in the bottom. At the time of design loading, the bottom fiber stress at midspan was 192 psi tension.

Although no edge restraint was used, there was no detection of horizontal movement of the lower edges during the test. A cantilever of 3 ft was used at the ends. A deflection of only ¾ in. was measured at the time of maximum loading.



$$I = \frac{b}{6d}\left[b^2 L^2 + d^2 h^2\right]$$

$$\bar{Y}_b = \frac{\frac{bL}{h+d}}{2}$$

$$S_b = \frac{I}{\bar{Y}_b}$$

Fig. 1—Formula for physical properties of folded plate

Although the member supported five times its design live load, no cracks were visible anywhere in the structure at the conclusion of the test.

It may be concluded from the results of this test that the prestressed folded plate, while presenting an outstanding appearance, also possesses impressive strength properties. The structural shape of the fold itself contributes much to this strength. Even though the integral parts of the folded plate contain no significant strength by themselves, these same flat slabs, when combined to form the folded member, yielded relatively large moments of inertia and section moduli. Formulas for determining the section properties of a folded plate are presented in Fig. 1.

The folded plate is perhaps in one of its most intriguing forms when used as long spanning roof members. Up to now, most of the folded plates constructed have been of ordinary reinforced concrete which necessitated expensive forming costs, as the members had to be cast-in-place.

The prestressing industry is now incorporating the economic principles of precast, prestressed concrete into the material saving aspects of the folded plate. The product may be mass-produced at a central manufacturing plant, thereby eliminating expensive form costs, since the individual plates may be cast directly on a flat prestressing bed.

SAVINGS

It is apparent that there are material savings involved by using a prestressed design, other than the savings gained through the elimination of high forming costs. Given a certain span and loading condition, a prestressed concrete folded plate will need a shallower section than a reinforced design because in the former, the bottom fibers are precompressed. Therefore, a prestressed folded plate is able to support a relatively greater superimposed load because of its ability to receive more tensile stress in its bottom fibers.

Once the initial cost of installing the prestressing bed is out of the way, forming costs become negligible. The flat slabs which comprise the folded plate may be cast side by side on a simple flat prestressing bed. This makes it possible to use relatively inexpensive end buttresses, in that the strands are all in one horizontal line, a short height above

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the bottom of the bed. The strands are deflected horizontally in the flat prestressing beds by the use of simple horizontal jacks, thereby reducing labor and material costs. The beds themselves need not be massive to resist high deflection forces.

Forms need not be built for each design. One 12 ft wide flat prestressing bed will supply the form work for an infinite number of designs. This is brought about by simply changing the angles of fold or the over-all dimensions of the folded plate. Thus, forming costs are greatly reduced as a result of the tremendous diversity of the single bed located at a central prestressing plant.

When the flat slabs are cast side by side, pick-up points may be easily provided by casting pieces of cross steel into the members. The two slabs may be picked up as a unit, allowing the desired angle of fold to be reached, at which time a clamp is placed on each end to prevent further folding.

The folded members do not have to be transported one at a time, but may be stacked on top of each other until load limits are reached. In most states, a folded plate with an over-all width of 8 ft or less may be easily transported without special permit.

The possibility of a machine that will place, vibrate, compress, form, and finish the flat slabs used in forming the folded plate is quite conceivable. An extrusion machine, developed by David Dodd, of Galatin, Tenn., is now producing 2 ft wide hollow core flat slabs and it is anticipated that the machine will be capable of serving the purposes of the folded plate in the near future. The machine will operate in a continuous process by laying the slabs out on a flat bed on which strands have previously been tensioned.

The savings which will be realized with this machine production are many. Particularly significant is the fact that the product may be stockpiled for immediate delivery. The flat slabs may be easily stored on top of each other.

HINGES

There are two methods which may be used in the forming of the hinges at the valleys and ridges of the folded plates. One method uses a pipe hinge assembly consisting of three short pieces of pipe together with a pin which has welded to it a washer or a nut which will not come loose. These assemblies are welded on 10 to 15 ft on centers to an angle iron which has been cast and anchored into the folded plate at 10 to 15 ft on centers.

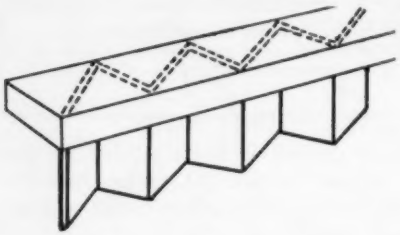


Fig. 2—Tie beam support for vertical load-bearing walls

The use of this type of hinge will necessitate a method of construction different from the second method which utilizes the cross steel to form the hinge. When the pipe hinge is used, the flat members are lifted from the bed and placed on temporary assembly frames located at a certain on-center spacing that will prevent deflection. Great care should be taken to align the frames accurately at the center line and toe, to insure that the folded member will be straight. Once the slabs are placed on the frames, the pipe hinge assembly is welded at the ridges. The valley hinge is welded in after the folded plates have been erected.

The hinges may also be formed with the cross steel cast into the flat slabs. The procedure for lifting the completed member from the bed has been mentioned. The joint is completed by grouting to the extended cross steel. An example design is included on p. 1320, and illustrates this type of hinge construction.

It is important that these joints be made completely watertight. Until further work is carried out no definite solution is offered. It is, however, conceivable that a galvanized iron or copper flashing may be used, with the areas between sprayed with waterproof plastic paints or epoxies.

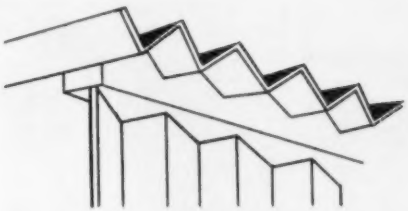
SURFACE FINISH

The folded design lends an attractive, unusual effect to a building, both as a roof and as a wall member. Complimenting this with a glass smooth finish definitely enhances its intriguing effect. Such a finish may be achieved by lining the forms with a neoprene or plastic sheet, or as an alternative the concrete may be coated with a smooth surface plastic.

BLAST RESISTANT WALL PANELS

The folded plate design may be used economically as an attractive wall panel. Used as a vertical member, (they can be horizontal) the folded member may be either load-bearing or non-load-bearing. In the case of the former, a cast-in-place reinforced concrete tie beam could be used at the tops. This tie beam supports the folded plate roof deck (Fig. 2 and 3).

Fig. 3—Tie beam support with folded plate roof deck



Concrete is ideally suited for blast resistant construction. It has high mass, and, therefore, possesses a large inertia. When used as a wall panel of any appreciable height, the folded plate posses a significant L/r ratio, a property which lends it the ability to become a highly blast resistant structure. Prestressing this member enables it to be resilient, or able to absorb a high overload, deflect, and recover without permanent deformation.

CONTINUITY

The folded plate acts as a simple beam, and can be analyzed as such. It follows that continuity could be a benefit to this type of design. While the steel requirements are obviously reduced because of the reduction in moment, the depth of section is also reduced for a constant span length. Continuity is used in the case of certain combined pretensioning and post-tensioning designs. The dead loads are carried by a pretensioned simple beam, and the live loads are carried by a post-tensioned continuous beam. Some continuous span designs may introduce excessive bottom fiber stresses over the supports. Such stresses may be relieved by placing an additional amount of field-placed concrete in the troughs of the folded plate at the supports.

COMPOSITE MEMBERS

In bridge and floor construction, a topping is cast-in-place on the folded plate, which acts as the "bare beam," in the composite design. The folded member is precast and prestressed, after which it is lifted into place. The reinforced decking is cast on plywood or corrugated metal forms. In some cases, it is advantageous to shore the bare beam while the decking is cast to reduce the bottom tensile stresses. The ultimate moment capacity of the composite member is not changed by shoring; however, and it should be left to the discretion of an experienced engineer whether or not the folded plate should be shored.

COMBINED POST-TENSIONING AND PRETENSIONING

It may be desirable to use a combination of pretensioning and post-tensioning for some of the longer, heavier members. Take, for example, a long span bridge or floor member. The folded plate would be pre-

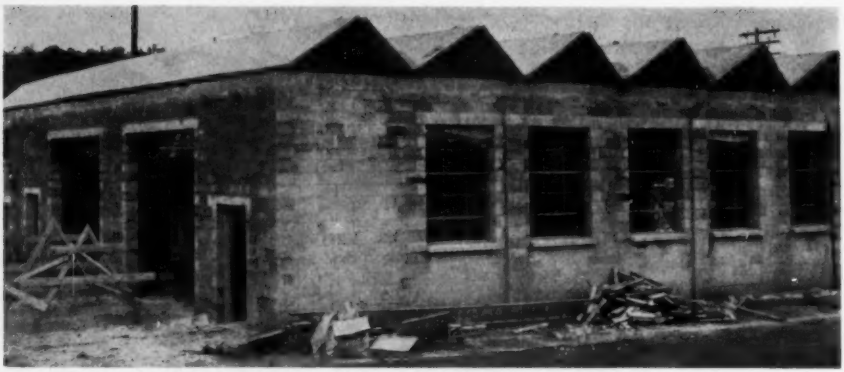


Fig. 4—Folded plate roof for shop building, Powder Springs, Ga.

tensioned to enable it to support its own dead weight as well as the load imposed by the decking. As the additional loads resulting from the entire bridge or floor structure are applied, the need for additional prestressing becomes apparent.

This post-tensioning is applied to the existing structure, and is usually applied by use of a draped conduit. At the time the decking is placed, diaphragms are also cast-in-place. The flexible conduit is located in the lower part of a diaphragm cast at midspan. Diaphragms are also cast at each end with the conduits so located in these end pieces as to provide less eccentricity than at midspan, where there is an acting moment.

DESIGN

The design of this proposed system does not follow the customary procedure of folded plate design. All the principal stresses are easily computed, and the design is simple.

The folded plate reacts as a simple beam in its long dimension, therefore, simple beam formulas may be used in computing fiber stresses, ultimate loads, cambers, and deflections. The physical properties of the folded plate may be computed from the formulas shown in Fig. 1.

Bending moments at the fold are ignored as being too low to be critical. The hinges at the ridges and valleys are not grouted until the structure is in its final place on the building. Therefore, the dead weight does not produce a bending moment in the joints. The thin slabs show a high degree of flexibility and transmit only small moments laterally.

A cast-in-place or precast diaphragm is added to each support to distribute the bearing over a large area. For this purpose, the diaphragms are triangular shaped, so as to obtain the largest bearing area and best load distribution possible.

Prestressing introduces a horizontal force component into the folded plates to counteract the diagonal tension, resulting in low shear stresses near the end supports. Bottom concrete at midspan is precompressed, thus assuring freedom of tensile cracks under design load.

A folded plate roof is used to cover a shop building for Leap Structural Concrete, Powder Springs, Ga. (see Fig. 4). Through the cooperation of E. G. Prentzas, chief engineer and plant manager for the Georgia firm, the design of these prestressed folded roof members is presented, along with the accompanying illustrations.

EXAMPLE (see Fig. 5)

Design span: 60 ft

Design criteria:

Allowable stresses: (5000 psi concrete)
Tension: 425 psi
Compression: 2250 psi
No stress allowed at end span in top fibers
Use 22—⅜ in. diameter strands (11 in each plate)
Superimposed live load: 30 lb per sq ft

Section properties:

Use equivalent rectangle theory:

$$\text{Area} = 2 \times 2.5 \times 60 = 300 \text{ sq in.}$$

$$b_2 = \frac{300}{40} = 7.5 \text{ in.} = \text{equivalent rectangle width}$$

$$S_2 = \frac{b_2 d_2^2}{6} = \frac{7.5 \times 1600}{6} = 2000 \text{ in.}^3$$

$$I_2 = S_2 \times \frac{d_2}{2} = 2000 \times 20 = 40{,}000 \text{ in.}^4$$

Prestress:

$$\frac{P_f}{A} - \frac{P_f e}{S_2} = 0$$

$$P_f = 22 \times 11{,}200 = 246{,}400 \text{ lb (assume 20 percent losses)}$$

$$\therefore \frac{P_f}{A} = \frac{246{,}400}{300} = 820 \text{ psi}$$

To make zero stress at top of ends, make $\frac{P_f e}{S_2} = 820$ psi

$$\therefore e = \frac{820\ (2000)}{246{,}400} = 6.66 \text{ in.}$$

Use $Y_s = 13.29$ in. $\quad \therefore e = 20.00 - 13.29 = 6.71$ in.

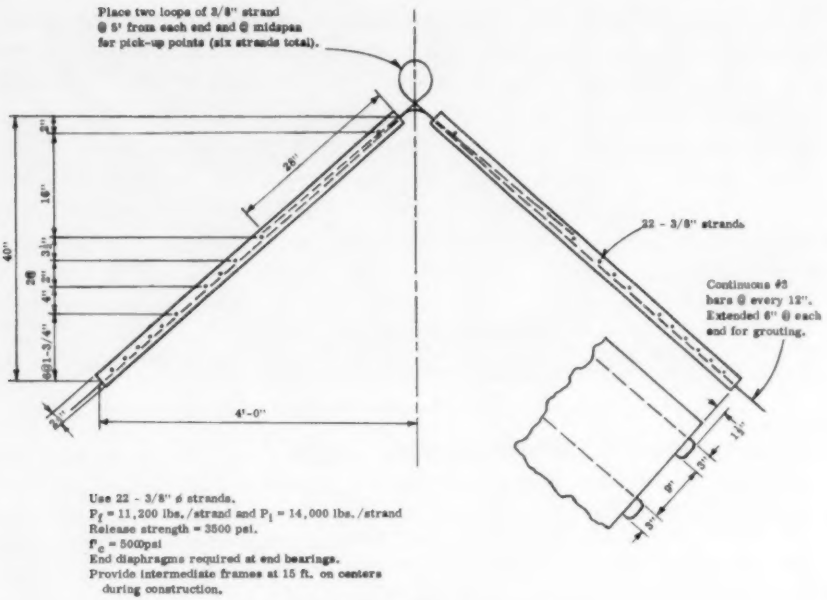


Fig. 5—Details of folded plate hinge

Stresses due to prestress:

$$f_b^{ps} = \frac{P_f}{A} + \frac{P_f e}{S_2} = 820 + 827 = 1647 \text{ psi}$$

$$f_t^{ps} = \frac{P_f}{A} - \frac{P_f e}{S_2} = 820 - 827 = -7 \text{ psi}$$

Stresses due to loads:

$$f^b_{DL} = \frac{M_{DL}}{S_2} = \frac{1.5\ (312.5)\ 3600}{2000} = \pm 844 \text{ psi}$$

$$f^b_{LL} = \frac{1.5\ (240)\ 3600}{2000} = \pm 648 \text{ psi}$$

Summary of stresses:

	At support		At midspan	
	Top	Bottom	Top	Bottom
P/A	820	820	820	820
Pe/S	−827	827	−827	827
f^{ps}	− 7	1647	− 7	1647
f^b_{DL}			+844	−844
Σ_i			+837	803
f^b_{LL}			648	−648
Σ_{ii}			+1485	+155

Concrete release strength:

$$f'_{ci} = \frac{1647}{0.6 \times 0.8} = 3500 \text{ psi}$$

Ultimate moment:

$$M_{DL} = 1.5\ (312.5)\ 3600 = 1{,}687{,}500 \text{ in.-lb}$$

$$M_{LL} = 1.5\ (240)\ 3600 = 1{,}296{,}000 \text{ in.-lb}$$

$$\text{Required } M_u = 5{,}370{,}300 \text{ in.-lb}$$

$$\text{Provided } M_u = P_u\ [d - 0.6\ P_u/bf_c'] = 12(20{,}000) \left[34.75 - 0.6\ \frac{12 \times 20}{5 \times 7} \right]$$

$$= 7{,}354{,}000 \text{ in.-lb}$$

Camber:

$$\Delta_c = \frac{18L^2}{EI}\ [1.17\,Pe - 1.25\,W_{DL}\,L^2]$$

$$= \frac{18\ (3600)}{40{,}000 \times 2.5 \times 10^6}\ [1.17\ (246{,}000)\ 6.71 - 1.25\ (312.5)\ 3600]$$

$$= 0.34 \text{ in.}$$

Shears:

$$V_{max} = 553\ (30) = 16{,}590 \text{ lb}$$

$$V_u = 1.8\ (16{,}590) = 29{,}862 \text{ lb}$$

$$V_c = 180\ bjd = 180\ (7)\ \tfrac{7}{8}\ (26.71) = 29{,}446 \text{ lb}$$

$$A_v = \frac{\frac{1}{2}\,(V_u - V_c)}{f_s jd} = \frac{\frac{1}{2}\,(416)\ 12}{40{,}000\ (\frac{7}{8})\ 26.71} = 0.00027 \text{ sq in per ft}$$

$$\text{Minimum } A_v = 0.0025bS = 0.0025\ (7)\ 12 = 0.21 \text{ sq in. per ft}$$

Use #3 at 12 in. for entire length

DESIGN SUMMARY

The zero stress specification in the top fibers, due to prestress, is primarily a conservative step taken to prevent a tensile stress from occurring at the ridge joint in the top of the folded member. Further research on the jointing of the folded plates will be necessary before a tensile stress will be confidently allowed at the joints of the folded plate. The tensile stress at the valley joints occurs only temporarily, as this stress is the result of the superimposed design loads.

Holding the stress due to prestress to a minimum also serves the purpose of keeping the camber down within allowable limits, as the eccentricity is given a maximum value.

The use of the equivalent rectangle theory gives results which are in close agreement with the formulas.

In calculating the ultimate moment capacity, only 12 strands were assumed to contribute, as the upper ten strands were considered to lay above or too close to the point of zero stress to be ultimately

TABLE 1 — COST ANALYSIS

Item	Unit cost	Quantity	Cost per sq ft
Concrete	$19.00 per cu yd	0.08 cu yd per ft	$0.183
Reinforcing steel	$ 0.11 per lb	275 lb per member	$0.063
Prestressing steel	$ 0.05 per ft	1362 ft per member	$0.142
Labor	$ 1.75 per hr	1.33 man-hr per sq ft	$0.240
Overhead and profit	(80 percent of materials and labor)		$0.502
Erection	$40.00 per hr	14 hr per 2400 sq ft	$0.230
	Total		$1.36 per sq ft

stressed. The ultimate load requirement is easily satisfied, although the assumption is conservative.

Referring to Fig. 5, attention is called to the spacing of the pick-up points, along with their development and the steel used. Two strands of ⅜ in. diameter prestressing steel are used for each pick-up point, of which there are three. It is necessary that the development be deep enough to prevent pull-out, and also that it be placed with respect to the prestressing steel as shown. This latter precaution will prevent the strand from pulling the concrete from the topside.

The reinforcing steel is extended at the joints to provide a bonding surface for the grouting. The top cross steel is extended through both slabs at the time they are precast on the flat beds. The same steel is extended at the bottom edges and bent as shown in Fig. 5.

An estimate of cost for the example design problem is summarized for study in Table 1. The total is based on cost per square foot of horizontally projected area. The result is slightly less than the cost for a prestressed double tee design for the same span.

CONCLUSION

The prestressed, precast folded plate offers tremendous potential for widespread use in buildings for roof, floor and wall construction, and in highway bridges.

Through prestressing, the folded plate is provided with the necessary properties to make it competitive with other structural members for long spans and heavy loads. Savings in formwork, labor and materials, along with such attractive features as standardization, availability, mass-production, and blast resistance assures the prestressed folded plate of an important place in the structural field.

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Title No. 57-67

Load-Deflection and Vibration Characteristics of a Multistory Precast Concrete Building

By JACK R. JANNEY and JOHN F. WISS

The load-deflection and vibration characteristics of a structure are closely associated. The use of high strength concrete in prestressed building elements along with composite construction can produce building components which have resonant frequencies at various stages of loading to which the human body is sensitive. Vibrations with low amplitude may be sensed easily if the resonant frequency of the structure is relatively high. This paper describes full scale load and vibration tests conducted on a multistory precast building. The degree of composite action was determined and found to be much more complete than normally considered in design. The vibration characteristics were determined and are discussed.

■ THE USE OF PRECAST CONCRETE structural members in the construction of multistory buildings is becoming of increasing interest to structural engineers. Precast members have been used for many years as roof and floor units and for decorative panels. However, designers have only recently begun to make use of precast members for the framework of multistory buildings as well as the roof, floor, and wall units.

The most effective method thus far devised to tie precast members together within a building is with cast-in-place concrete designed to act with the precast members to form a composite system. This coupled with the high strength concrete normally used in precast structural units often results in floors and floor supporting members with greater stiffness than is experienced in conventionally reinforced concrete or steel frame buildings.

The vibration characteristics of a structure are a function of the product EI. Thus, high values of modulus of elasticity of concrete combined with large values of moment of inertia which are a consequence of composite action can result in natural frequencies in structures which are in a range where small amounts of motion are easily sensed by the human body.

ACI member **Jack R. Janney** is a principle in Engineers Collaborative, Chicago, and heads their structural laboratory in Des Plaines, Ill. He spent 6 years with Portland Cement Association laboratories where he initiated research on prestressed concrete. He has been a member of ACI-ASCE Committee 323, Prestressed Reinforced Concrete, since 1950 and is chairman of ACI-ASCE Committee 712, Precast Structural Concrete Design and Construction.

John F. Wiss is one of the principals of Engineers Collaborative. He has specialized in vibration problems for over 15 years and is a recognized authority on the seismic and structural effects of blasting. He is the author of a number of articles on the analysis and control of foundation vibrations and was among the first to use air springs to isolate machinery vibrations.

A four story warehouse in Dade County, Fla., was constructed of precast prestressed beams supported by reinforced concrete columns with precast, prestressed joists spanning between the beams. A 3 in. cast-in-place deck, designed to contribute structurally to the floor system through composite action with the beams and joists, was placed on each floor. The floors were designed to carry a live load of 300 lb per sq ft.

After construction a noticeable vibration, produced by walking across the unloaded second, third, or fourth floors, was detected. Concern regarding the effectiveness of the composite action between the cast-in-place and precast concrete was expressed as a consequence of this phenomenon.

Load tests on the structure were proposed to determine if the precast elements and cast-in-place concrete were composite and to determine the vibration characteristics of the floors.

The test program consisted of two parts:

1. Load tests during which strains and deflections were measured
2. Vibration measurements of displacement and frequency during various load conditions

LOAD TEST

The load tests were conducted in two parts. First, a fork lift truck was loaded with slabs of zinc until all the weight was on the two front wheels. The total load was 10,600 lb or 5300 lb per wheel. The loaded truck was moved to various positions on the fourth floor to develop moments and shears at the desired sections. Second, a uniform load of 245 lb per sq ft was placed over a portion of the floor to compare the behavior of the structural components with that observed from the concentrated load tests.

SR-4 strain gages were used to measure concrete strains. These measurements served a dual purpose: first, to derive concrete stresses at points of maximum moments, and second, to determine the neutral axis from the strain diagrams at the concrete cross section at the points of instrumentation.

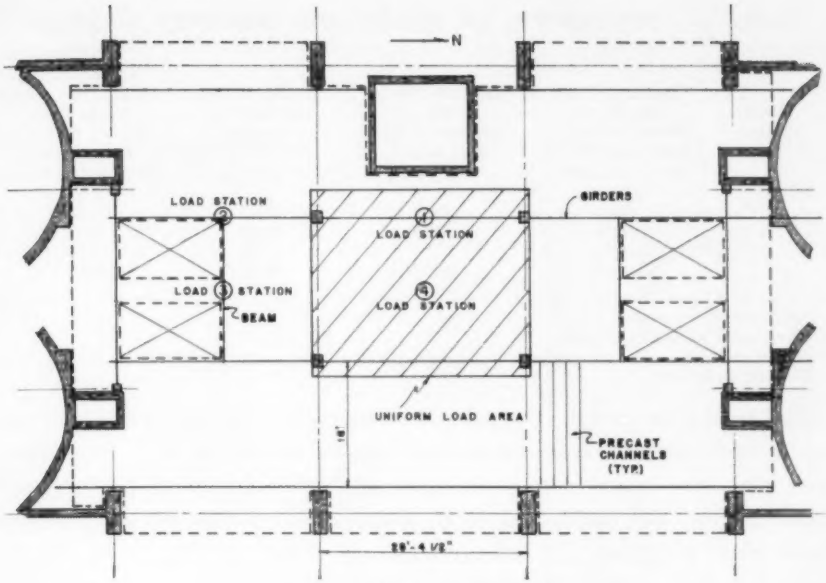

Fig. 1—Plan of fourth floor

The positions on the fourth floor which were instrumented with SR-4 strain gages and dial deflection gages are shown in Fig. 1.

LOAD TEST RESULTS

Strain gages were placed at Load Stations 1, 2, 3, and 4 (see Fig. 1) so that the neutral axis of the cross section at each station could be determined. If the neutral axis of a member subject to bending is known, the effective cross section may be determined. Thus, it was possible to determine how much of the cast-in-place concrete acts with the precast elements to resist bending.

The loaded fork lift truck which weighed 10,600 lb was placed at each load station. Strains and deflections were measured and then the load was moved to the next load station.

Fig. 2 shows the location of the four strain gages on the beam cross section at Load Station 1. The beam loaded as shown with two concentrated loads developed strains which placed the neutral axis 26 in. from the bottom. Thus, with the neutral axis at this point, 72 in. of the 3-in. cast-in-place floor must be acting as the top flange of a T-beam. The shaded area of the cross section was assumed to be effective in the original calculations. It may be seen that the beam is appreciably

TABLE 1 — COMPARISON OF DESIGN AND MEASURED STRESSES*

Load station	Bottom fiber stress, psi (from Wright's design calculations) LL only	Bottom fiber stress, psi (measured from test load)	Ratio of test moment to design moment (simple span assumed)	Bottom fiber stress, psi (test stress projected to correspond to design moment)
1	1680	397‡	0.82	485
2	1230	115§	0.18	645
3	1330†	225§	0.53	430
4	1410	880‡	0.82	1075

*All stresses are tensile
†Dust collector load included
‡Uniform load
§Concentrated load

stiffer than anticipated. The calculated moment of inertia was 78,800 in.^{4} while that derived from measured strains is 168,300 in.^{4} Therefore, the composite beam is 2.16 times as stiff as assumed in the original floor design. Similarly, the strain data at Load Stations 2, 3, and 4 have been analyzed and the results are shown in Fig. 3, 4, and 5. Each I-beam became a T-beam by virtue of the contribution of a considerable portion of the 3-in. cast-in-place floor. Load Stations 1 and 4

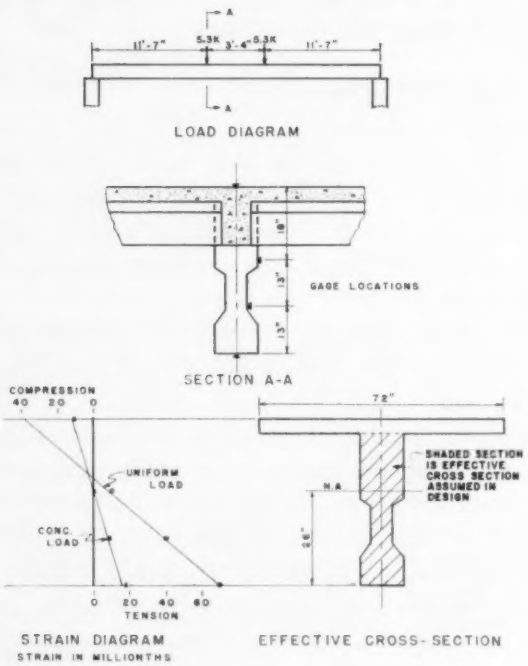

Fig. 2 — Strains at Load Station 1

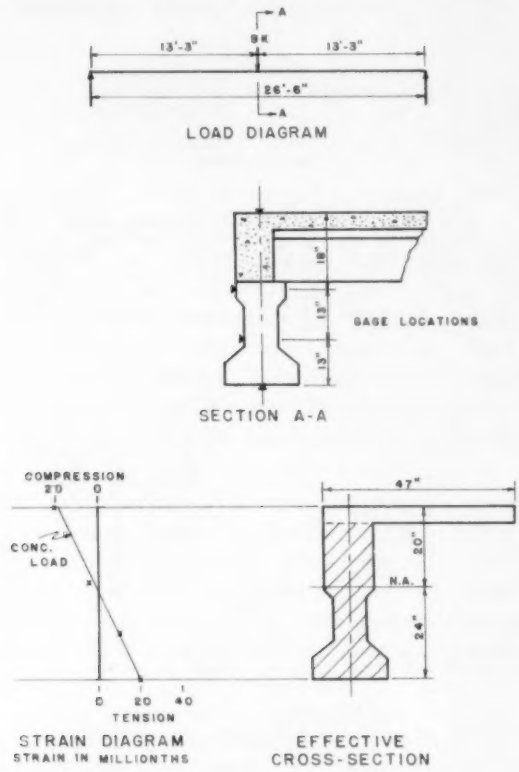

Fig. 3 — Strains at Load Station 2

were the only ones at which both concentrated load and uniform load data were obtained.

The comparison between the design live load stresses and those interpreted from strain measurements is shown in Table 1. The test moments were not equal to design moments so the test stresses were extrapolated linearly to correspond to live load design stresses. These extrapolated stresses were less than predicted because the measured moment of inertia was greater than design calculations and also because some negative moment was developed at the supports. The design calculations were based on simple span moments. Assuming simple spans, and using the measured moment of inertia of the cross

TABLE 2 — COMPARISON OF CALCULATED DEFLECTIONS BASED ON SIMPLE SPAN AND MEASURED DEFLECTIONS

Load Station	Deflection based on simple span calculation, in.	Measured deflection, in.
1	0.007	0.004
2	0.006	0.004
3	0.007	0.008
4	0.027	0.019

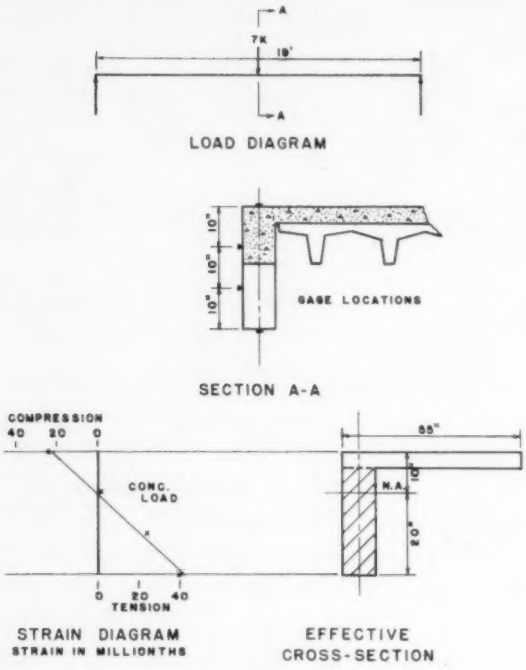

Fig. 4 — Strains at Load Station 3

section at each load station, the deflections have been calculated and are presented in Table 2. These values are compared with measured deflections. It may be seen that the measured deflections are smaller than those predicted from simple span considerations in each case except for the beam framing a cavity in the floor (Load Station 3). The reduction in deflections coupled with the vibration behavior of the floor discussed hereinafter leads to the conclusion that the reinforcement over the beam and joint supports served as negative reinforcement thereby reducing the positive moments at midspan. The beam framing the cavity could not develop any negative moment on the ends because a gap existed between the beam end and the web of the supporting beam.

An impact hammer was used to estimate the quality of the concrete at points of instrumentation over the structure. The impact hammer readings indicated that the concrete strength for the precast elements was in the neighborhood of 7500 psi. The modulus of elasticity of this concrete was assumed as 5,500,000 psi for converting strain to stress. The compressive strength of the concrete for the topping was assumed to be 5000 psi based on impact hammer readings. The modulus of elasticity for the topping concrete was assumed to be 3,500,000 psi.

VIBRATION TESTS

The vibration investigation was directed toward:

1. Measuring the dynamic deflection (vibration) and frequency of oscillation of the floors under "walking conditions"
2. Determining the natural frequency response of the floors to impact loads
3. Measuring existing intensities and modes of vibration produced by the operation of the machinery located on the fourth floor

One of the dust collector fans located on the fourth floor was unbalanced to a considerable extent at the time of the tests. Thus, a significant source of steady-state vibration was available. This was used to obtain data on the dynamic response of the floor under the dead load, concentrated load, and at various stages of the uniform test loading.

It is considered appropriate to review briefly certain established vibration fundamentals and relationships which are pertinent to these investigations.

A structural member, when subjected to a sudden force or impact, will vibrate at its natural frequency (cycles per second). This characteristic frequency of the member is inversely proportional to the square root of the deflection produced in the member by all loads carried (including its own weight). Therefore, if a static load increases the deflection of a member by a factor of 4, the natural frequency will *decrease* by a factor of 2.

When a periodic disturbing force is imposed on a structural member, the resulting amplitude of vibration (displacement) depends on the intensity of the disturbing force and on the ratio of the frequency

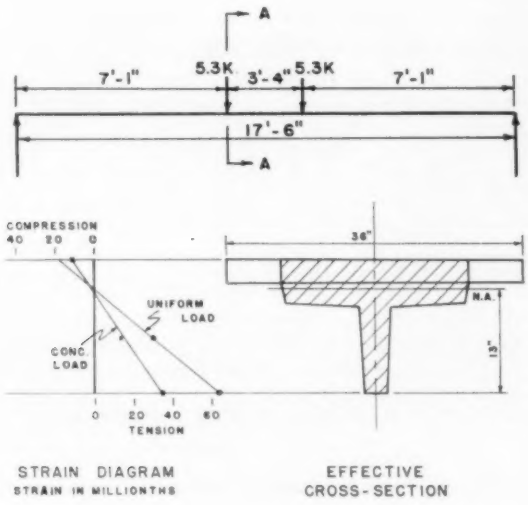

Fig. 5 — Strains at Load Station 4

of the force to the natural frequency of the member. If this ratio is 1.0, a condition of resonance exists and the member oscillates through a large amplitude. As the frequency ratio decreases or increases, the amplitude of the oscillation is reduced.

The instruments used for vibration measurements were portable blast and vibration seismographs. These instruments provide a magnified recording of the vibratory displacement in three mutually perpendicular directions, versus time. A photostatic reproduction of typical vibration patterns recorded during this investigation is shown in Fig. 6. The top trace is fore-aft (longitudinal) horizontal motion, the center trace is up-down (vertical) motion, and the bottom trace is left-right (transverse) horizontal motion. During periods of no agitation the traces

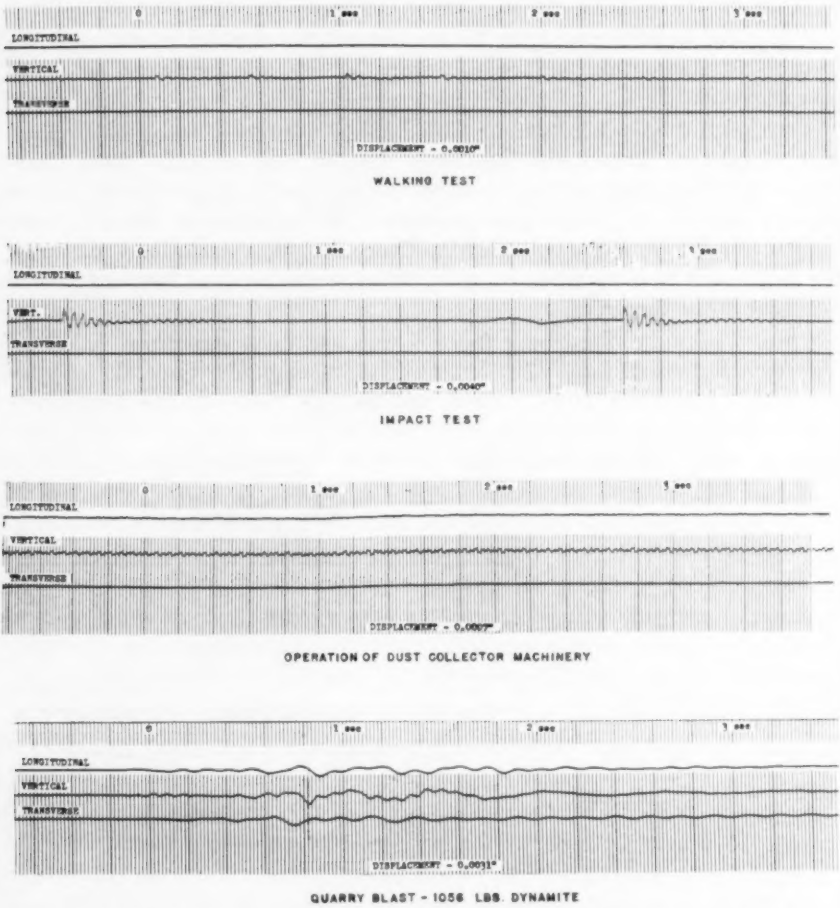

Fig. 6—Typical vibration patterns (center of fourth floor of packhouse)

are straight parallel lines. During agitation, the lines become a series of peaks which are exact magnified reproductions of the vibrations. When the agitation ceases, the traces become straight and parallel.

The magnification of these instruments is 50. Timing lines are placed on the record (crosswise of the traces) at intervals of 0.02 sec.

TABLE 3 — DEFLECTIONS CAUSED BY WALKING

Floor	Longitudinal, mils	Vertical, mils	Transverse, mils	Frequency, cycles per sec
1	0.1	0.1	0.1	100.0
2	0.1	0.9	0.1	20.0
3	0	1.0	0	20.0
4	0	1.0	0	20.0

Displacements are reported for each of the three mutually perpendicular components recorded. For this report, the units of displacement measurements are mils.* All values are single amplitude; for total movement multiply the displacements by 2. Frequency is measured in cycles per second.

Walking tests

These tests consisted of measuring the response at the center of the several floors of the structure caused by a man (150 lb) walking normally across the floor. The amount of deflection caused by normal walking is reported in Table 3; the frequency of the vertical oscillation is also given.

It is apparent that the upper floors of the structure are identical in response while the first floor which is supported directly on earth is more rigid.

It is appropriate to mention that the vibration caused by a 150 lb man walking resulted in a displacement of 1 mil, while the static load of 10,600 lb produced 19 mils deflection. The nonproportionality is due to the dynamic nature of the "load" applied to the floor during the walking tests.

Natural frequency tests

For purposes of determining the natural frequency of the floor panels, impact forces were produced by dropping heavy objects on the center of the floor while the seismographs were recording resulting movements. The results of these tests reveal the natural frequencies of the floor panels to be as shown in Table 4.

TABLE 4 — NATURAL FREQUENCIES OF FLOOR PANELS

Floor	Natural frequency, cycles per sec
1	100
2	20
3	20
4	20

The natural frequencies of the several floors are the same as those

*A mil is 0.001 in.

TABLE 5 — DECREASE IN NATURAL FREQUENCY WITH INCREASE IN LOAD

Uniform superimposed load	Natural frequency, cycles per sec
0	20.0
31,000 lb (48 lb per sq ft)	15.2
66,000 lb (100 lb per sq ft)	12.5
140,000 lb (214 lb per sq ft)	9.1
160,000 lb (245 lb per sq ft)	9.5

frequencies recorded during the "walking tests." The impact tests were made to verify that the same frequencies prevailed during larger deflections of the floor panels (10.0 mils).

Identical natural frequency tests were made on the fourth floor during the progressive application of the uniform load. The decrease in the natural frequency with the increased load is shown in Table 5. The slight increase in natural frequency above the 140,000 lb uniform load is caused by the higher loading being outside the main beam (see Fig. 1), which reduced the static deflection at the center of the panel because of continuity over the support. Therefore, the natural frequency was increased slightly.

Machinery vibration

To establish the prevailing level of the vibration produced by the machinery located on the fourth floor of the building, a grid was established and measurements made from 28 positions on the floor. The results of these measurements are presented in Fig. 7. Only the vertical vibration is shown since the two horizontal components in all cases were less than 0.2 mil. Dust collectors 1 and 2 (and associated machinery) were operating normally during this test series; units 3 and 4 were not operating. These were subsequently checked and were found to be in balance. The No. 1 dust collector fan was the unit previously mentioned as having a relatively high degree of unbalance.

It is apparent from Fig. 7 that the greatest single source of vibration is the No. 1 dust collector fan. The response of the floor system is as expected with the panel at the center of the floor moving up when the section of the floor supporting the No. 1 dust collector fan is moving down. This further confirms the observation made that the temperature steel in the floor slab provided some moment continuity over the beams. The peak value registered between the No. 1 and No. 2 dust collectors is not caused by the unbalanced fan. This is apparent from the fact that this portion of the floor is vibrating at 30.0 cycles per sec while the rest of the floor is vibrating at the fan rotational speed of 15.0 cycles per sec.

The application of the 10,600 lb concentrated load at the center of the fourth floor had no significant effect on the vibration at this point caused by the normal operation of the machinery. The same load applied at the base of the No. 1 dust collector fan however increased

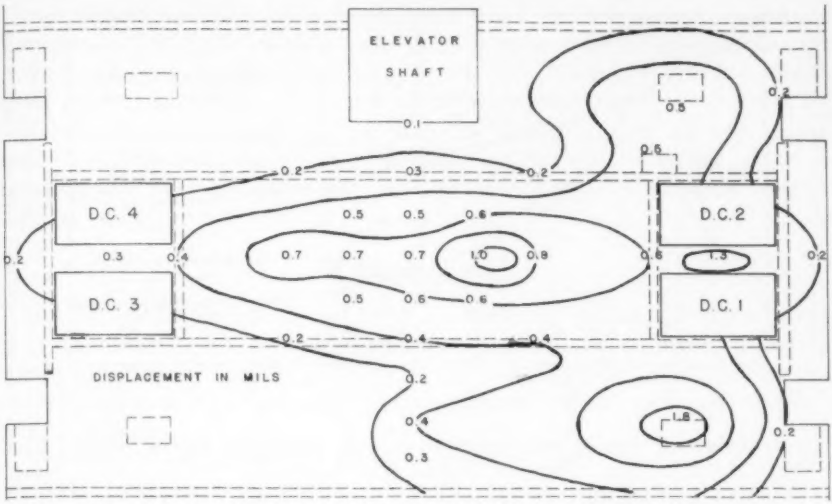

Fig. 7—Vertical vibration of fourth floor (during operation of Dust Collectors 1 and 2)

the vibration at the base of the unit from 1.8 to 4.2 mils. This resulted because the concentrated load increased the static deflection of the floor supporting the unit, thereby bringing the natural frequency of the floor closer to the operating frequency of the fan and producing a "near resonant" condition.

The uniform load was applied by stacking bags of cement over the center area of the fourth floor. Recordings of the vibration at the center of the floor panel (Load Station 4) produced by the No. 1 dust collector fan were made as the loading progressed. The results of these measurements are shown in Table 6.

Fig. 8 illustrates the response of the fourth floor to vibration of the machinery. This shows the variations of vibratory displacement in the center of the floor and the natural frequency of the floor with the degree of loading. The operating frequency of the No. 1 dust collector fan is also indicated. The agreement with theory is illustrated by the common point of natural frequency, operating frequency, and maximum displacement occurring at the partial load of 31,000 lb (as applied in this case). The physical significance of these

TABLE 6 — VIBRATORY DISPLACEMENT OF LOADED FLOOR

Test load, lb	Vibratory displacement, mils
0	0.7
31,000	3.0
44,000	2.1
80,000	1.1
140,000	0.7
160,000	0.7

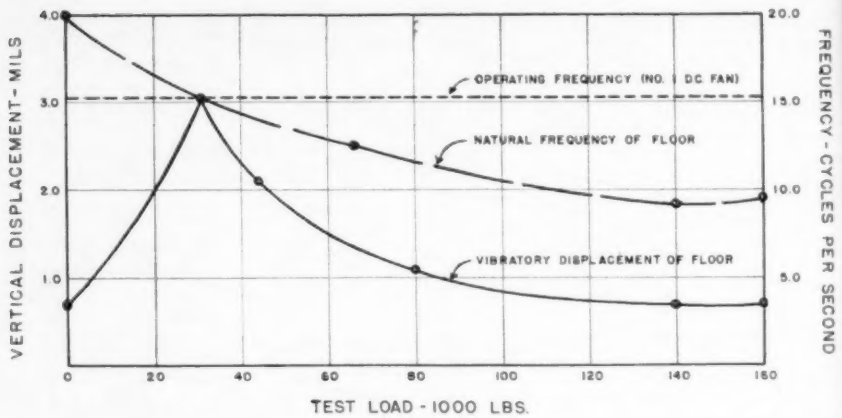

Fig. 8—Effect of loading on vibratory response (center of fourth floor of packhouse)

data is that as the floor is loaded, the vibration in the floor increases to a maximum value (corresponding to 31,000 lb total load); as the floor is further loaded, the vibration will decrease until it is at the same intensity as exists when the floor is unloaded.

Human response

With reference to the walking tests it is significant to discuss briefly the response of the human body to vibration. The body is an excellent detector of all forms of stimulation; in many cases it is more sensitive than the finest instruments. It is not reliable however as a measuring device. Evaluation of vibration depends on the displacement, the frequency, the direction of the vibration, and the position of the body. Controlled laboratory tests on human subjects have revealed that the body is most sensitive to vertical vibration when the subject is standing, and most sensitive to horizontal vibration when lying down. For purposes of illustration the following Table 7 is presented for the two different floor frequencies recorded during these walking tests. The values were taken from extensive experimental data by Reiher and Meister at the Institute of Technology, Stuttgart, Germany (1931). In these tests many subjects of various ages and of different backgrounds were vibrated at controlled displacements and frequencies, and independently evaluated the vibration based on their

TABLE 7 — HUMAN RESPONSE TO VIBRATION

	Displacement, in.	
	At 20 cycles per sec (upper floors)	At 100 cycles per sec (first floor)
Perceptible	0.00010	0.00003
Objectionable	0.00080	0.00015*
Uncomfortable	0.00170*	0.00022*

*Relative position of displacement measured during walking tests

own sensations. The data so obtained have been checked by other investigators and are considered to be the most reliable information on human response to vibration. Table 7 lists the values for vertical vibration, subject standing.

It is apparent from the above that the walking tests on the upper floors produced intensities of vibration which are relatively high on the human response scale, and classify in a higher zone than similar activity on the first floor (on earth fill).

CONCLUSION

The load tests demonstrated that an assembly of precast units properly designed to act with cast-in-place topping result in an effective composite floor system which acts as if it were monolithic. The extent of composite action was considerably greater than that assumed for design purposes.

The degree of composite action combined with the high strength concrete used for the precast members resulted in a greater stiffness of structural elements and consequently higher natural frequency than is normally encountered in a cast-in-place reinforced concrete structure with similar dimensions. The natural frequencies of the unloaded second, third, and fourth floors were within a range in which the human body is sensitive to relatively small motion. Thus, the insignificant vibration caused by walking was sensed by persons standing on the floor whereas the same amount of motion on a floor of lower natural frequency would not thus be detected.

The importance of preventing the natural frequency of structural elements in a building from being in resonance with any operating machinery was emphasized in these tests.

The effectiveness of providing for continuity by placing reinforcement in the cast-in-place topping over supports was also demonstrated. Even though such provisions were not intended, the temperature steel in the topping resulted in appreciable negative moment capacity in the floor slab over the main girders.

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Title No. 57-58

Torsional Strength of Prestressed Concrete Members

By PAUL ZIA

Reported are the results of torsional tests of 68 pretensioned and plain concrete members consisting of rectangular, T- and I-sections. Some of the specimens also contained web reinforcement in the form of ties.

For the rectangular and T-sections, the test results compare closely with the predicted strength by a modification of Cowan's failure criterion. According to this criterion, which is noted as a close approximation of Mohr's failure theory, the optimum uniform prestress is 70 percent of the concrete cylinder strength. Corresponding to this optimum value, the torsional strength of a member is increased to 281 percent of that of the similar plain concrete member if the cylinder strength is 6000 psi.

For the I-sections, the test results exceed considerably the predicted strength. Explanation of this discrepancy is offered as being the consequence of stress redistribution after the initial cracking. However, it is suggested that the usable strength of such a section be taken as the cracking moment, since the cracks due to torsion remain open on unloading and thus permanently damage the concrete member.

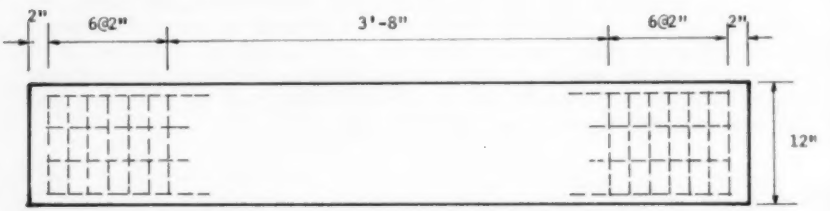
■ TORSION IN SIMPLE CONCRETE STRUCTURES is generally a secondary effect which seldom causes difficulties to the designer. Nevertheless, in the design of members such as a spandrel beam supporting a cantilevered balcony, a beam curved in plan, a spiral stairway, or a skewed arch, torsion is a factor that must be considered. In precast concrete construction, the handling of the slender precast members also requires, at times, the assessment of their torsional capacity.

Investigation of torsional strength of concrete members dates as far back as 1903. Most of these early investigations (see Reference 14 for a comprehensive bibliography) have dealt with plain and reinforced concrete members of circular and rectangular sections. Limited number of tests were also carried out on members of T and L section. A summary of these researches can be found in the appendix of Reference 14. More recently, Cowan and Armstrong[1-4] contributed to this subject a great deal. Their theoretical and experimental investigations extended to combined bending and torsion of rectangular reinforced and post-tensioned concrete members. The number of tests in each case was, however, rather limited. The most extensive torsion test of prestressed concrete members reported to date was by Humphreys.[5]

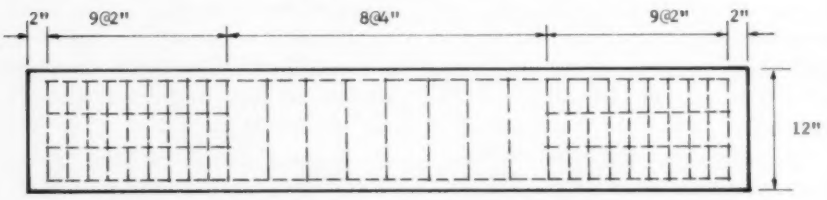
ACI member **Paul Zia** is assistant professor of civil engineering, University of Florida, Gainesville. Dr. Zia has for years been an active consulting engineer in reinforced and prestressed concrete construction. He is a member of ACI Committee 338, Torsion.

He tested 94 post-tensioned rectangular members of varying width-depth ratio from 1 to 4. The prestressing force varied from 0 to 6000 psi. Four of the specimens were eccentrically prestressed and the remainder were uniformly prestressed.

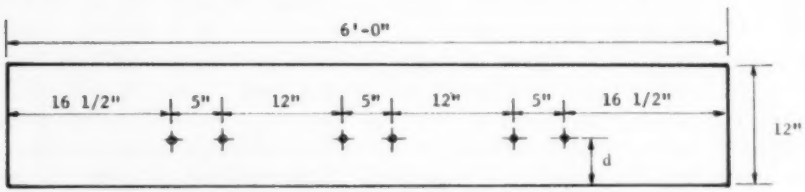
The present investigation deals with pure torsion of pretensioned concrete members of rectangular, T, and I section. Some of these members also contain a nominal amount of web reinforcement. The purposes of this study are: to examine critically the various theories of failure; to develop a simple and conservative failure criterion for the determination of the torsional strength of prestressed concrete members; to study the effect of the web reinforcement on the behavior and strength of these various members under torsion.



Elevation of Series R (All reinforcements are ϕ2 bar)



Elevation of Series RW (All reinforcements are ϕ2 bar)



Elevation Showing Location of Strain Gage Points

Fig. 1—Elevations of R and RW series specimens

Notation

The symbols used in this paper are generally defined when they are introduced. The most frequently used symbols are listed below for convenient reference.

b = width of a rectangle
d = depth of a rectangle
M_T = twisting moment or torque
G = modulus of rigidity, psi
θ = angle of twist per unit length
K = torsion constant = $\frac{M_T}{G\theta}$
τ = apparent torsional strength
τ_{max} = maximum torsional stress
τ_o = torsional strength of plain concrete
f = normal stress due to effective prestress, psi
f_c' = compressive strength of concrete, psi
f_t' = tensile strength of concrete, psi

EXPERIMENTAL PROGRAM

Description of test specimens

The testing program consisted of 84 specimens of which 68 were tested under pure torsion and 16 were instrumented for determination of loss of prestress. The specimens were of three different cross sections, namely, rectangular, T, and I. Corresponding to each cross section there were two series, one with and one without web reinforcement. Table 1 shows the schedule of all the specimens and Fig. 1 through 6 show their details. Referring to the specimen number, the letter symbol "R" stands for rectangular section, "T" for T-section, "I" for I-section, "W" for web reinforcement, and "P" for plain concrete. The

TABLE 1 — SCHEDULE OF TEST SPECIMENS

Series R		Series RW		Series T		Series TW		Series I		Series IW	
Specimen No.	Type of test	Specimen No.	Type of test	Specimen No.	Type of test	Specimen No.	Type of test	Specimen No.	Type of test	Specimen No.	Type test of
OR1	T*	ORW1	T	0.25T1	T	0.25TW1	T	0.75I1	T	0.75IW1	T
OR2	T	ORW2	T	0.25T2	T	0.25TW2	T	0.75I2	T	0.75IW2	T
OR3	T	ORW3	PL			0.25TW3	PL			0.75IW3	PL
OR4	PL†										
OR5	PL										
2R1	T	2RW1	T	2.25T1	T	2.25TW1	T	3I1	T	3IW1	T
2R2	T	2RW2	T	2.25T2	T	2.25TW2	T	3I2	T	3IW2	T
2R3	T	2RW3	PL			2.25TW3	PL			3IW3	PL
2R4	PL										
2R5	PL										
2.5R1	T	2.5RW1	T	2.75T1	T	2.75TW1	T	3.5I1	T	3.5IW1	T
2.5R2	T	2.5RW2	T	2.75T2	T	2.75TW2	T	3.5I2	T	3.5IW2	T
2.5R3	T	2.5RW3	PL			2.75RW3	PL	3.5I3	PL	3.5IW3	PL
2.5R4	PL										
2.5R5	PL										
RP1	T	RW1	T	TP1	T	TW1	T	IP1	T	IW1	T
RP2	T	RW2	T	TP2	T	TW2	T	IP2	T	IW2	T
RP3	T	RW3	T	TP3	T	TW3	T	IP3	T	IW3	T
RP4	T	RW4	T					IP4	T	IP6	T
RP5	T	RP7	T					IP5	T	IP7	T
RP6	T	RP8	T								
		RP9	T								

*Torsion test
†Prestress loss control

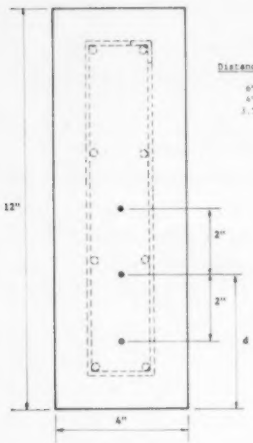


Fig. 2—Cross sections of R and RW series specimens. Prestressed with three 7/16-in. strands initially tensioned to 19,750 lb for R series and 18,900 lb for RW series

number preceding the letter symbol represents in inches the eccentricity of the prestressing force.

Materials and fabrication of specimens

The concrete used in the specimens was transit-mixed. The design mix per cubic yard of concrete is as follows:

Cement (Type III)	705 lb
Brooksville (Florida) Limerock (specific gravity = 2.40, FM = 6.44)	1956 lb
Sand (specific gravity =2.56, FM = 2.39)	1005 lb
Water	31 gal
Pozzolith "8"	7.5 lb

The average slump of all batches of concrete used was 6 in. Standard 6 x 12 in. concrete cylinders for each series of specimens were prepared and cured under the same atmospheric conditions as the specimens. Table 2 shows the results of cylinder strength and modulus of elasticity of the concrete used on various series of specimens both at the time of transfer of prestress (4 days) and at 28 days.

TABLE 2—SUMMARY OF CONCRETE STRENGTH AND MODULUS OF ELASTICITY

Series	At transfer of prestress (4 days)		At 28 days	
	f_c', psi	E_c, psi*	f_c', psi	E_c, psi*
R	6390	4.08×10^6	7200	4.14×10^6
RW	6025	3.95×10^6	7480	4.48×10^6
T	5900	4.38×10^6	7250	4.68×10^6
TW	5900	4.38×10^6	7250	4.68×10^6
I	5650	3.96×10^6	7050	4.45×10^6
IW	4200	4.04×10^6	5550	4.72×10^6

*Initial tangent modulus from stress-strain curve of cylinder.

The specimens were all pretensioned and fabricated by a prestressed concrete products manufacturer. The 7/16-in. strands were tensioned individually by a power-operated hydraulic jack. The required elongation of each strand was carefully measured and the amount of tension was further checked by the pressure gage on the pump. When the concrete was 4 days old, prestress was transferred to the specimens by simultaneous burning of the strands near the abutments at both ends of the casting bed. When the testing was carried out, the specimens were at least 7 days old.

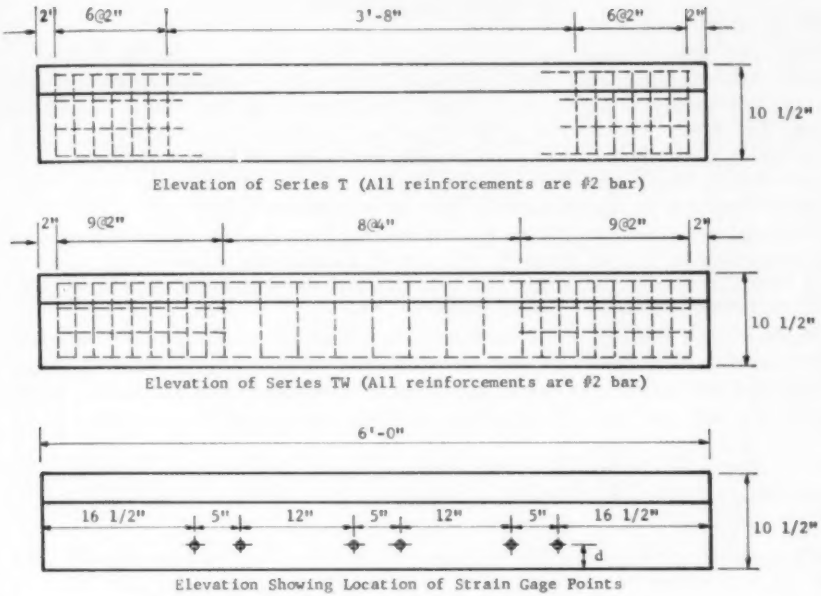


Fig. 3—Elevations of T and TW series specimens

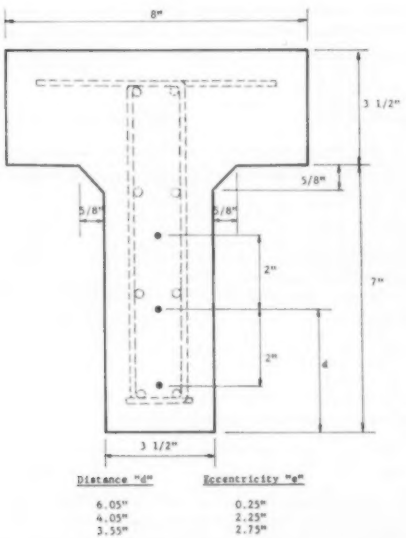


Fig. 4—Cross sections of T and TW series specimens. Prestressed with three 7/16-in. strands initially tensioned to 18,900 lb

Test arrangement and procedure

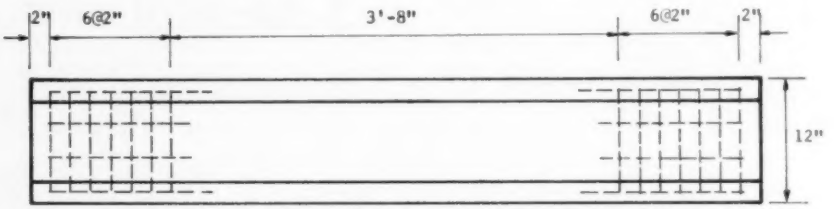
The tests were carried out in a specially built torsion machine shown in Fig. 7. The machine has a fixed end and a movable head which rides on two tracks. The torque arms are 40 in. long. The capacity of this machine is 60,000 in-lb. The angle of twist was measured by the mirror-scale method.

During the first several trial experiments, specimens often cracked prematurely in the chucks which held them in the machine. This difficulty was later overcome by using a ⅛ in. thick lead sheet as a cushion.

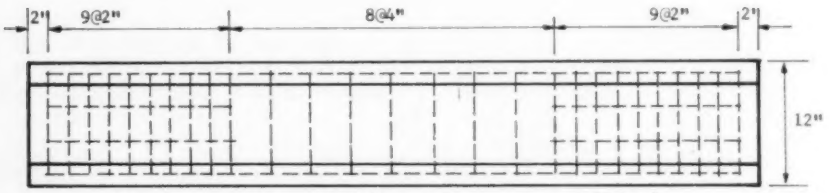
Each specimen was loaded in two cycles. In the first cycle of loading, the specimen was loaded to about 70 percent of its ultimate capacity in three or four load increments. This loading cycle allowed the opportunity to detect any necessary readjustment of the instrumentation.

In the second loading cycle, the specimen began to exhibit yielding when it was loaded beyond about 80 percent of its ultimate strength. As the torque approached the ultimate moment, the yielding of the specimen was quite fast so that the loads and the mirror-scale readings were recorded simultaneously every 3 to 5 sec.

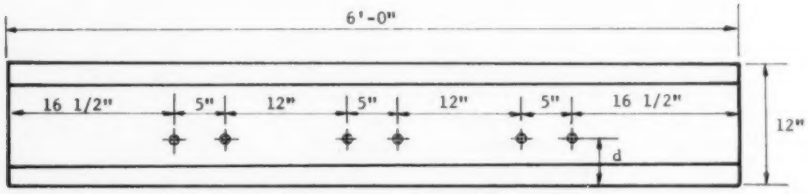
For every group of specimens tested on the same date, at least two cylinders were tested to ascertain the concrete strength of the specimens at the time of test.



Elevation of Series I (All reinforcements are #2 bar)



Elevation of Series IW (All reinforcements are #2 bar)



Elevation Showing Location of Strain Gage Points

Fig. 5—Elevations of I and IW series specimens

ANALYTICAL STUDIES

Torsional stress

The analytical solution of St. Venant's elastic torsion theory for a rectangular section is well known[6] but such a solution for I- and T-sections is not mathematically feasible. As a recourse, numerical solutions using the method of finite differences were chosen in this investigation. A detailed discussion of this method and finite difference expressions for various derivatives can be found in Reference 7.

Because of symmetry, only one quarter of the rectangular and I-section and one-half of the T-section need be considered in the analysis. One in. square mesh for the rectangular and I-sections and 7/8 in. square mesh for the T-section were used in writing the difference equations. This choice of mesh size leads to 12 linear simultaneous equations for the rectangular section, 16 for the I-section, and 32 for the T-section. The solutions of the simultaneous equations were obtained with the aid of an IBM 650 computer. The maximum stresses and torsion constants for the three different sections are summarized in Table 3. It is seen that for the rectangular section the results by the numerical procedure is in close agreement with St. Venant's analytical solution.

A simple approximation of the torsion constant and the maximum torsional stress for a narrow rectangle were proposed by Bach[8] where the torsion constant is

$$K = \frac{M_T}{G\theta} = (\tfrac{1}{3})\, b^3 d \qquad (1)$$

and the maximum torsional stress is

$$\tau_{max} = \frac{M_T b}{K} \qquad (2)$$

If a section is composed of a number of narrow rectangles, such as I and T, the torsion constant will be approximately the sum of the torsion constants of the component rectangles. Hence

$$K = \Sigma\, (\tfrac{1}{3})\, b^3 d \qquad (3)$$

The maximum stress will occur at the middle of the side having the

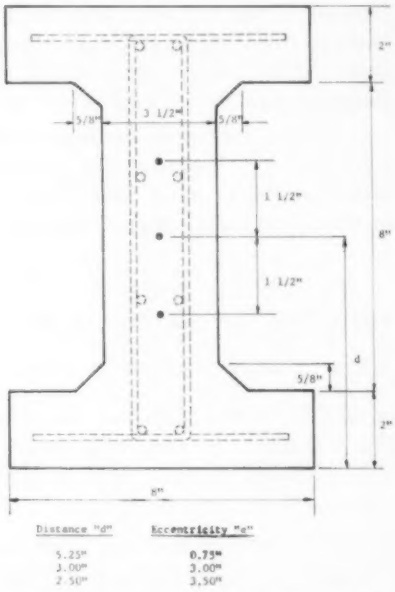


Fig. 6—Cross sections of I and IW series specimens. Prestressed with three 7/16-in. strands initially tensioned to 18,900 lb

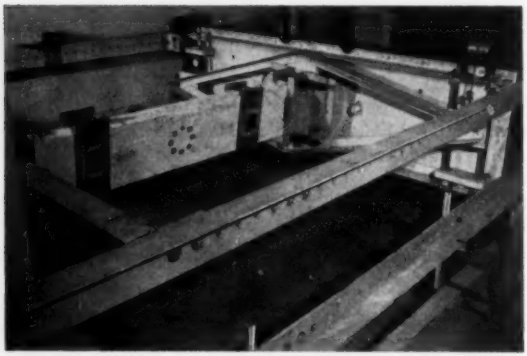


Fig. 7—Torsion machine

largest thickness of the component rectangles and is equal to

$$\tau_{max} = \frac{M_T t}{K} \quad (4)$$

in which t is the largest thickness of the component rectangles.

It can be seen in Table 3 that the results by Eq. (3) and (4) differ considerably from the results by the finite difference method. This difference is due to the fact that Bach's approximation neglects two effects on the value of the torsion constant, namely, the effect of the ends of the flanges and the effect of the junction of the flange and the web. There is a reduction in the approximate value of the torsion constant due to the end effect and a gain due to the junction effect.

Lyse and Johnston[9] made a refinement to Bach's approximate method, considering these end and junction effects with the aid of the membrane analogy using soap film. According to their results, the gain in torsion constant due to the junction effect is

$$\Delta K_1 = \alpha D^4 \quad (5)$$

where D is the diameter of the inscribed circle at the junction, and

$$\alpha = 0.094 + 0.070(r/m)$$

in which r is the radius of the fillet and m is the thickness of the flange. The reduction in torsion constant due to each flange end is

$$\Delta K_2 = 0.10504\, b^4 \quad (6)$$

where, as before, b is the thickness of the flange.

Applying these corrections, Eq. (5) and (6), to Eq. (3), we have the torsion constants of the three sections as follows: For the rectangular section,

$$K = \tfrac{1}{3} b^3 d - 0.21008\, b^4 \quad (7)$$

TABLE 3 — SUMMARY OF TORSION ANALYSIS BY VARIOUS METHODS

Cross section	Item	Finite difference method	St. Venant solution	Bach's method	Lyse-Johnston method
Rectangular Section	Stress at middle of long side	$0.01964M_T$	$0.0195M_T$	$0.0156M_T$	$0.0197M_T$
	Stress at middle of short side	$0.01306M_T$			
	Torsion constant K	199.29	202	256	203
I-Section	Stress at fillet	$0.019M_T$		$0.0223M_T$	$0.0187M_T$
	Stress at middle of web	$0.0183M_T$			
	Stress at middle of flange	$0.0165M_T$			
	Torsion constant K	192		157	206
T-Section	Stress at fillet	$0.0152M_T$		$0.0164M_T$	$0.0165M_T$
	Stress at middle of web	$0.0107M_T$			
	Stress at middle of flange	$0.0123M_T$			
	Torsion constant K	280		214	233

For the I-section ($D = 3.80$ in. and $\alpha = 0.0334$),

$$K = \Sigma \tfrac{1}{3} b^3 d + 2\alpha D^4 - 0.42016\, b^4$$
$$= \Sigma \tfrac{1}{3} b^3 d + 0.2668\, D^4 - 0.42016\, b^4 \quad (8)$$

For the T-section ($D = 4.875$ in. and $\alpha = 0.1165$),

$$K = \Sigma \tfrac{1}{3} b^3 d + \alpha D^4 - 0.31512\, b^4$$
$$= \Sigma \tfrac{1}{3} b^3 d + 0.1165\, D^4 - 0.31512\, b^4 \quad (9)$$

For the maximum torsional stress, Lyse and Johnston suggest

$$\tau_{max} = \frac{M_T(w + 0.3\, r)}{K} \quad (10)$$

where w is the web thickness and r is the radius of fillet. The results by Eq. (7) through (10), as given in Table 3, are seen to be in good agreement with the results by finite difference method.

Criteria of failure

When a uniaxially prestressed member is under twist, there exists a state of biaxial stress. To evaluate the strength of the member, one must establish a criterion of failure. Several theories of failure for concrete under combined stresses have been proposed. These theories are represented graphically in the form of failure envelopes in Fig. 8(a). If the state of stress in concrete is represented graphically by Mohr's circle, then the failure of concrete is impending if the circle becomes tangent to the failure envelope.

Fig. 8a—Failure theories

Rankine's maximum stress theory assumes that a material will fail under any condition of loading if one of the principal stresses at any point reaches the value of the limiting stress as determined by simple tension or compression test of the material regardless of what the stresses may be on other planes through the point. The failure envelopes of this theory are, therefore, two straight lines, parallel to the τ-axis and tangent to circles C_1 and C_2, Mohr's circles representing pure shear and simple compression, respectively.

Coulomb's internal friction theory assumes that failure of a material is caused by a sliding action along a certain plane within the material. The resistance to this sliding is composed of a constant shearing strength, similar to cohesion, and an internal friction which is proportional to the normal stress on the sliding plane. When the shearing stress overcomes this combined resistance, failure occurs. This theory is graphically represented by a straight line drawn tangent to C_1 and C_2 as the failure envelope.

This internal friction theory was later generalized by Mohr. It assumes that the resistance to shear on the sliding plane is a function of the normal stress acting on the plane. Hence the failure envelope is a curve as shown in Fig. 8(a). The exact shape of the envelope must be determined experimentally by testing the material in various combinations of stress and by drawing a series of the largest stress circles and then the envelope. Such envelopes were obtained by Karman and Böker for their tests on marbles[10,11] and by Richart, Brandtzaeg, and Brown for their tests on concrete.[12] It must be pointed out that Mohr's theory neglects the influence of the intermediate principal stress. While this influence is significant in the case of triaxial stress, it is rarely important for members under biaxial stress.

Cowan suggested the combination of the maximum stress and the internal friction theories as a dual criterion of failure for concrete.[2] His suggestion was based on the observation of two distinct modes of failure of concrete, cleavage and shear fracture. The failure envelope of this theory therefore consists of two parts: the vertical line same as Rankine's envelope and also the line tangent to Circle C_2 and having an inclination of 37 deg. This angle is assumed to be the angle of internal friction of average concrete.

A comparison of Cowan's envelope with Mohr's envelope would reveal that the former is, in effect, an approximation of the latter. How-

ever, there is a fundamental difference between the two. Cowan's criterion implies that there is a definite state of stress at which failure of concrete changes abruptly from the pure cleavage type to the shear-fracture type. Mohr's theory, on the other hand, implies a gradual transition from pure cleavage failure to pure fraction failure. It must be observed that during this transition stage, (which is represented by the cases having the stress cricles tangent to the envelope to the left of the τ-axis) the normal stress on the plane of failure is tensile. This tensile stress, contrary to the compressive stress, greatly reduces the formation of debris due to the sliding action along the failure plane. (The formation of debris is an indication of shear fracture failure.) So it is possible that a combination of cleavage and sliding failure could be misinterpreted as a pure cleavage failure.

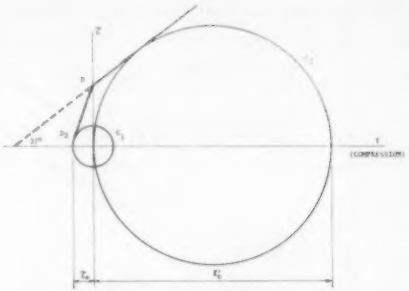


Fig. 8b—Modified Cowan Theory of failure

In an attempt to devise a unified and yet simple criterion, a modification to Cowan's theory is therefore proposed. In Fig. 8(b), line DD_2 is drawn from the point of intersection of the Cowan envelope with the τ-axis and is tangent to Circle C_1. This modified envelope is seen to be a closer approximation to Mohr's envelope than Cowan's original envelope, and this theory will be referred to, hereafter, as the Modified Cowan Theory.

It should be noted that all the criteria discussed above depend on two distinct properties of the concrete; the compressive strength and the torsional strength, τ_o. The assessment of the torsional strength is usually difficult for two reasons: (1) it requires elaborate setup of a torsion machine; (2) it necessitates test specimens in the form of a thin-walled cylinder. However, since a plain concrete member under pure torsion will fail in diagonal tension, the torsional strength is therefore equal to the tensile strength. From the extensive test results by Gonnerman and Shuman,[13] the tensile strength of concrete appears to be related to the compressive strength as follows (Fig. 9);

$$\tau_o = f_t' = 0.68(f_c')^{3/4} \quad \ldots (11)$$

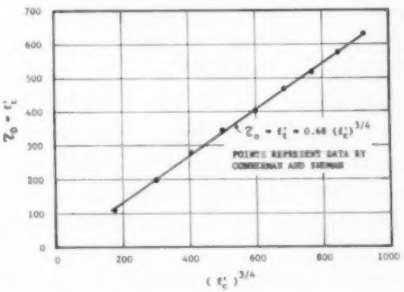


Fig. 9 — Concrete tensile-compressive strength relationship

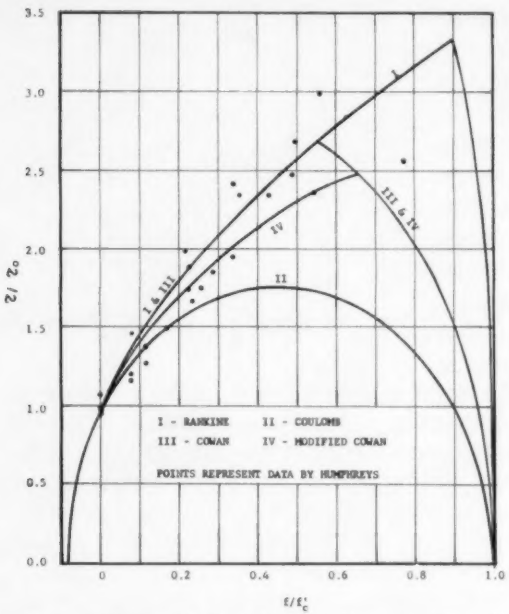


Fig. 10 — Humphreys' test results compared with various failure theories. $f_c' = 7870$ psi, $\tau_o = 740$ psi

Expressions of the interaction curves for the various criteria have been derived [14] and these curves are shown in Fig. 10. It can be seen that the various criteria differ from each other to a considerable measure. For concrete having a compressive strength of 7870 psi and a torsional strength of 747 psi, the optimum compression is 90 percent of f_c' according to Rankine's theory, 46 percent of f_c' according to Coulomb's theory, 55 percent of f_c' according to Cowan's theory, and 65 percent of f_c' according to the Modified Cowan Theory.

Also shown in Fig. 10 are the test results of Humphreys[5] mentioned earlier. Each point on the graph represents the average results of three or more identical tests. Humphreys reported that the over-all average of the maximum torsional stress of his control plain concrete specimens is $\tau_o = 747$ psi and the average compressive strength of the concrete is 8730 psi as determined from the 4-in. cubes. In plotting Humphreys' data, the cube strength of 8730 psi was converted to the equivalent 6 x 12 in. cylinder strength by multiplying a conversion factor of 0.9.[15] These experimental evidences indicate decidedly that Coulomb's theory is not in agreement with the test results. Both Rankine's and Cowan's theories seem to err on the danger side and the Modified Cowan's Theory compares most favorably with the test data for the lower compression range. As for the higher compression range, there is not sufficient data to justify a choice among the various theories.

Fig. 11 shows the apparent torsional strength of concrete under various degrees of compression with the compressive strength of the concrete

as a parameter. These curves are prepared according to the Modified Cowan Theory and Eq. (11). The validity of these curves will be further substantiated by the author's own test results.

EXPERIMENTAL RESULTS AND DISCUSSION

General behavior of test members

The typical moment-twist curves for the rectangular, T-, and I-sections are shown in Fig. 12 to 14. In all cases it is seen that the members behaved elastically up to about 75 percent of their ultimate moments. Both the plain and the prestressed concrete members without web reinforcement failed abruptly after slight yielding. However, the members with web reinforcement, both prestressed and non-prestressed, exhibited considerable yielding before failure. This delayed failure indicates the decided toughness of the members containing web reinforcement.

At failure the nonprestressed members, whether with or without the web steel, have helical cracks at an angle of about 45 deg to their longitudinal axes. The cracks are well defined, showing the cleavage type of failure. On the other hand, the prestressed specimens both with and without web steel rupture more violently. The directions of the cracks depend on the magnitude and the eccentricity of the prestressing force. The larger the prestressing force, the smaller is the angle of

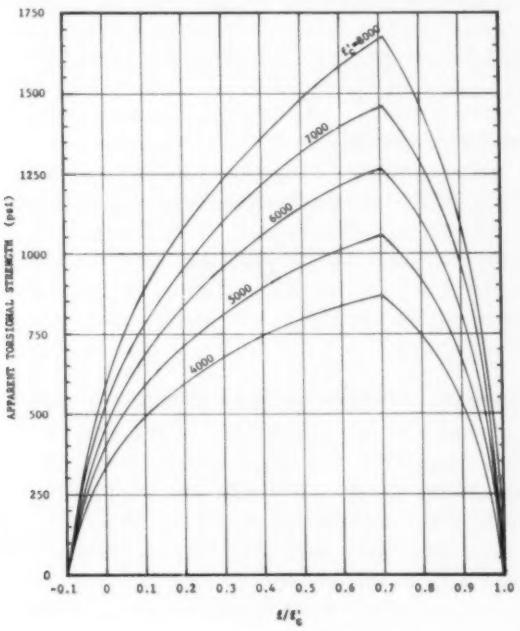


Fig. 11—Apparent torsional strength versus f/f_c'

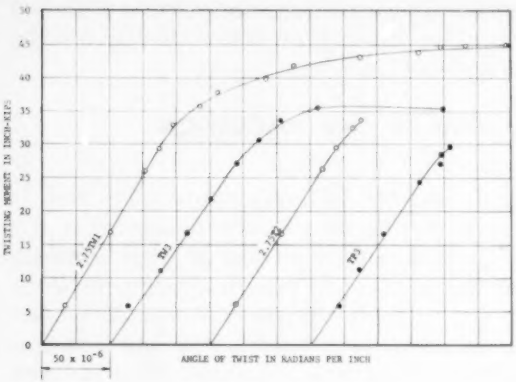


Fig. 12 — Typical torque-twist curves for R and RW series

inclination of the crack. For the eccentrically prestressed members, the crack usually forms at a point above the center of gravity of the prestressing steels and at a greater inclination. Then it propagates toward the prestressing steel with decreasing slope. This pattern suggests that, as the crack propagates, the normal stress due to the prestressing force on the uncracked portion of the member increases and thus reduces the inclination.

Furthermore, visual observations reveal that at the onset of failure, the cracks of the prestressed specimens have a tendency to form a helix. As soon as a definite crack forms on one side of the specimen, it penetrates into the specimen and leads to a destructive compression-sliding failure on the opposite side. Fig. 15 to 20 show the different types of specimens after failure.

Modulus of rigidity

The modulus of rigidity may be obtained in two ways: (1) from the slope of the stress-strain curve; and (2) from the slope of the moment-twist curve. The results from these two approaches are comparable and the presence of prestress and web reinforcement appear to have little effect on the magnitude of the modulus of rigidity. As an over-all average, the modulus of rigidity may be taken as 0.4 of the modulus of elasticity.

Ultimate strength of plain concrete members

Table 4 shows a summary of the results of the plain concrete sections. The maximum torsional stress computed from the ultimate moments according to the finite-difference method (see Table 3) are seen to exceed the values predicted by Eq. (11). Several early investigators have explained that the elastic theory overestimates the maximum torsional stress because of the plasticity of the concrete and advocated the use of the plastic torsion theory. However, Cowan[16] indicated that

for concrete with a plasticity ratio less than 0.5, the effect of stress redistribution was not significant. (Plasticity ratio is defined as the ratio of plastic strain to total strain.) For the results at hand, the comparison between the measured and predicted torsional strength is fair for the R and T series, but the differences between the measured and predicted strength of I series are indeed too great to be accounted for by the plastic action of concrete. This great difference is believed to be due to a secondary behavior of the I-section after the first cracking.

The web of the I-sections used in this investigation is thicker than the flanges and, therefore, the torsional stresses are much higher along the sides of the web than in the flanges. Thus, the first cracking of the I-section under tension begins in the web and spreads toward the flanges. The less stressed flanges, however, tend to arrest these cracks and the general breakdown does not immediately follow the initial cracking. Rather a redistribution of stress takes place in the flanges changing from the torsional shear to lateral bending shear. As a limiting condition, the external torque will be resisted by the internal couple formed by the equal and opposite lateral shear forces in the two

TABLE 4—TEST RESULTS OF PLAIN CONCRETE MEMBERS

Specimen No. (1)	f_c', psi (2)	Measured M_{ult}, in-kips (3)	τ_o, psi, from M_{ult} (4)	τ_o, psi, from Eq. (11) (5)	(6) = (4)/(5)
R series					
RP1	6000	28.55	562	463	1.21
RP2	6200	26.36	519	474	1.09
RP3	6350	24.24	476	483	0.99
RP4	6350	27.44	540	483	1.12
RP5	6400	28.04	551	485	1.14
RP6	6400	32.04	630	485	1.30
RP7	7180	28.24	550	529	1.04
RP8	6500	29.76	583	490	1.19
RP9	6950	31.24	612	516	1.19
T series					
TP1	6590	33.65	512	495	1.04
TP2	6785	32.10	490	506	0.97
TP3	6880	29.24	445	511	0.87
I series					
IP1	6210	39.61	755	475	1.59
IP2	6700	42.63	811	501	1.62
IP3	6700	38.92	742	501	1.48
IP4	6700	43.51	830	501	1.66
IP5	6700	42.93	817	501	1.63
IP6	4750	34.90	664	388	1.71
IP7	5270	35.20	670	420	1.60

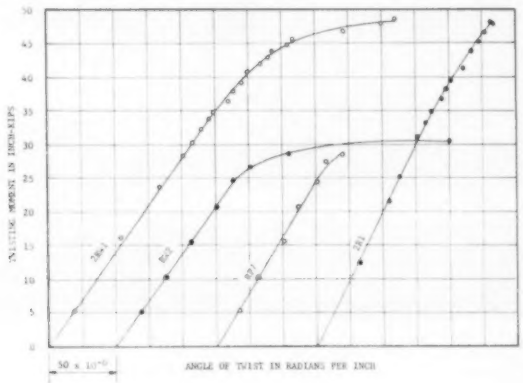


Fig. 13 — Typical torque-twist curves for T and TW series

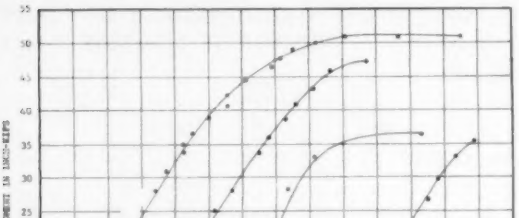


Fig. 14 — Typical torque-twist curves for I and IW series

flanges. Such a secondary behavior does not exist with the rectangular and T-sections where the first cracking precipitates immediate and complete failure.

Ultimate strength of prestressed members

The theoretical and measured ultimate torque of all prestressed sections are shown in Table 5. Knowing the normal stress on a section due to prestress, one may obtain the apparent torsional strength from Fig. 11 which has been prepared according to the modified Cowan failure criterion and the empirical expression for the tensile strength as shown by Eq. (11). With this apparent torsional strength, the theoretical torque of the each section is computed according to the results of elastic analysis as shown in Table 3. It should be noted, however, that the torsional capacity of an eccentrically prestressed section is

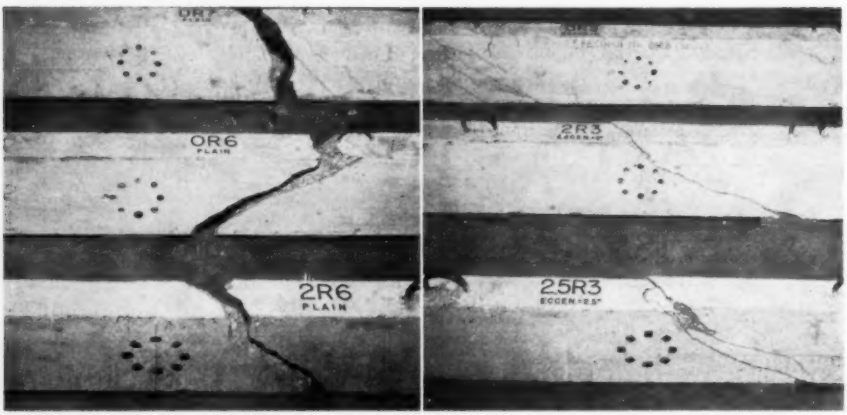


Fig. 15—Torsional failure of R-series specimens, (left) plain concrete, (right) prestressed concrete

governed by the strength of concrete at the top fiber even though the torsional stress at this point is considerably less than that at the middle of the sides of the web. This is because of the greater increase in the apparent torsional strength at the latter location due to the presence of higher prestress.

The theoretical and measured torques compare closely for the T series and first two groups of the R series. The third group of the R series, those with 2.5-in. eccentricity, shows substantially higher strength than the theoretical value which is based on the first cracking at the top fiber. This increase in strength after cracking is possible because of the increase in normal stress due to prestress caused by the reduction in net cross section of the member after cracking.

The theoretical torques of the I-series are seen to be much lower than the measured torques. To explain this discrepancy, two different types of behavior of the I-section after the initial cracking must be brought out. For the prestressed I-section with small eccentricity (Specimens 0.75I1 and 0.75I2), the initial cracking would begin in the web, leaving two flanges to resist the applied torque in the same manner as the I-section of plain concrete discussed in the previous section. For the I-sections with considerable eccentricity of prestress, the initial cracking would begin at the top fiber and the cracks then propagate into the web, leaving the uncracked portion in the form of a T. The normal stress on this uncracked portion of the I-section would immediately increase because of the reduction in net cross section and also the reduction in eccentricity of the prestressing force with respect to the centroid of the net section. This transformation of the eccentrically prestressed I-section into a prestressed T-section enables the member to sustain higher torque than that corresponding to the first cracking. The I- and T-sections used in this investigation have equal web thickness and almost the same flange size. It is interesting

TABLE 5 — TEST RESULTS OF PRESTRESSED CONCRETE MEMBERS

Specimen No. (1)	f_c', psi, at test (2)	Effective prestress, kips (3)	Ultimate torque, in-kips: Measured (4)	Ultimate torque, in-kips: Theoretical (5)	(6) = (4)/(5)
R series					
OR1	8050	48.87	44.20	47.80	0.93
OR2	8050	48.87	46.50	47.80	0.97
OR3	8200	48.40	46.72	48.10	0.97
2R1	8500	50.00	47.90	46.20*	1.04
2R2	8540	50.00	51.04	46.40*	1.10
2R3	8540	50.00	49.24	46.40*	1.06
2.5R1	8540	48.37	45.64	37.00*	1.23
2.5R2	8540	48.37	47.24	37.00*	1.28
2.5R3	8540	48.37	43.04	37.00*	1.17
T series					
0.25T1	6590	50.76	51.12	53.50	0.97
0.25T2	6590	50.76	51.98	53.50	0.97
2.25T1	6785	47.92	35.20	36.40*	0.97
2.25T2	6785	47.92	32.92	36.40*	0.91
2.75T1	6785	47.10	33.96	32.40*	1.05
2.75T2	6880	47.00	33.16	33.00*	1.00
I series					
0.75I1	4750	51.59	46.60	33.00	1.41
0.75I2	4750	51.59	46.10	33.00	1.40
3I1	5270	49.90	45.22	24.00*	1.88
3I2	5270	49.90	50.78	24.00*	2.12
3.5I1	6210	49.98	54.11	20.00*	2.70
3.5I2	6210	49.98	53.33	20.00*	2.66

*Strength of concrete at top fiber governs

TABLE 6 — RESISTING MOMENT OF WEB REINFORCEMENT

Specimen	b', in.	d', in.	p, in.	A_v, sq in.	M_T, in.-kips by Eq. (12)
R series	2	10	4	0.05	13.2
T series	1.75	9	4	0.05	10.4
I series	1.75	10	4	0.05	11.55

to note that the measured ultimate torques of the eccentrically prestressed I-sections are close to the measured ultimate torques of the T-sections (Specimens 0.25T, 0.25T2).

This unique behavior under torsion of the prestressed concrete I-section after initial cracking brings about additional strength which may be quite substantial. However, from the structural designer's viewpoint, the usable strength of a prestressed concrete member under torsion can only be considered as the torque corresponding to the initial cracking since the cracks formed in a member due to torsion do not close on unloading. So, the increased strength of the I-section beyond the initial cracking can be regarded as only an added safety factor.

Ultimate strength of members with web steel

A theory of torsion of a rectangular reinforced concrete member was advanced by Cowan.[1] On the assumption of elasticity, Cowan derived the total internal strain energy of a reinforced concrete member as consisting of the tensile strain energy in the steel and the compressive strain energy in the concrete. By equating this internal strain energy to the external work done by the twisting moment, he obtained an

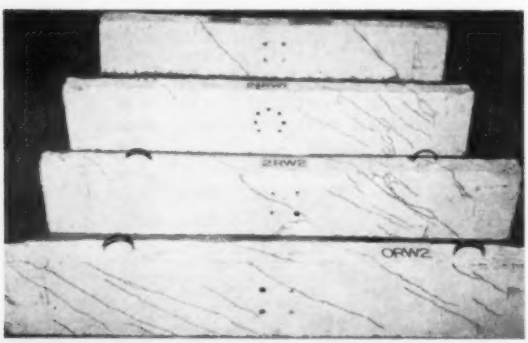


Fig. 16 — Torsional failure of RW-series specimens

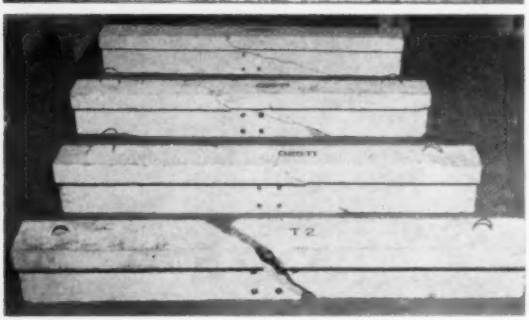


Fig. 17 — Torsional failure of T-series specimens

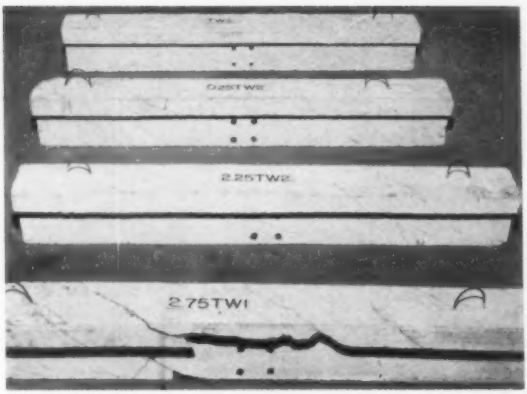


Fig. 18 — Torsional failure of TW-series specimens

expression for the twisting moment of a reinforced concrete member when the permissible stress in the concrete has been exceeded. If the web reinforcement is in the form of ties, the resisting moment of the web reinforcement is

$$M_T = \frac{1.6\, b'\, d'\, A_v\, f_v}{p} \qquad (12)$$

where

A_v = cross sectional area of *one* web reinforcement
b' = width of the web reinforcement cage
d' = depth of the web reinforcement cage
p = pitch of the web reinforcement
f_v = stress in the web steel

TABLE 7 — COMPARISON OF EXPERIMENTAL AND THEORETICAL ULTIMATE TORQUE OF MEMBERS WITH WEB REINFORCEMENT

Specimen No. (1)	f_c' psi, at test (2)	Avg measured ultimate torque, in.-kips (3)	Avg ultimate* torque, in.-kips (no web steel) (4)	Torque due to web steel, in.-kips (5)	Theoretical ultimate torque, in.-kips (6) = (4) + (5)
			RW series		
ORW1,2	7180	54.04	40.60	13.20	53.80
2RW1,2	6950	51.97	40.10	13.20	53.30
2.5RW1,2	6950	52.94	37.00	13.20	50.20
RW1,2,3,4	6950	29.81	30.51	13.20	(43.71)
			TW series		
0.25TW1,2	6590	52.32	51.55	10.40	(61.95)
2.25TW1,2	6785	43.85	34.06	10.40	44.46
2.75TW1,2	6880	48.70	33.56	10.40	43.96
TW1,2,3	6590	30.71	30.95	10.40	(41.35)
			IW series		
0.75IW1,2	4750	52.10	46.35	11.55	57.90
3IW1,2	5270	58.74	48.00	11.55	59.55
3.5IW1,2	5270	60.10	45.50	11.55	57.05
IW1,2,3	5270	36.55	34.20	11.55	(45.75)

*From test results shown in Table 5 adjusted by direct proportion for proper value of f_c' before average is taken.

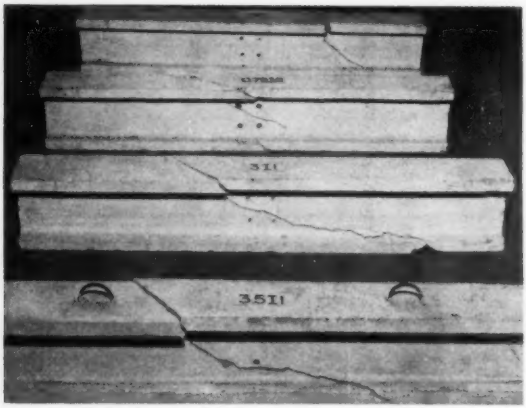


Fig. 19 — Torsional failure of I-series specimens

In addition to this vertical web reinforcement, there must be an equal volume of longitudinal steel to resist the horizontal component of the diagonal stress. The cross-sectional area of this longitudinal steel is

$$A_L = \frac{2\sqrt{2}A_v(b' + d')}{p} \quad \text{.....(13)}$$

It is noted from the moment-twist curves of all test members containing web steel that the curves become horizontal for a considerable amount of rotation before failure. This behavior indicates that the web steel is yielding prior to the failure of the test members. So, the stress in the web steel is assumed as 33,000 psi at ultimate torque. The data and the resisting moment of the web reinforcement in the three different sections may be found in Table 6.

The measured and theoretical ultimate torques of all test members with web steel are shown in Table 7. It is noted that in all three series the nonprestressed specimens with web steel have, for all practical purposes, the same strength as the similar specimens without web steel. The presence of the web reinforcement only increases the toughness of the specimen but not the strength. This lack of strength is believed to be due to the fact that the horizontal component of the diagonal stress cannot be resisted by the four longitudinal bars which are located at the corners of the narrow ties where the stresses are low. Since nothing resists the horizontal component of the diagonal stress, the vertical ties cannot develop their potential strength.

This reasoning does not apply, however, to the prestressed members for which the measured and the theoretical ultimate torques compare closely, with one exception—the group of TW series having 0.25 in. eccentricity of prestressing force. In all these prestressed members, even though the longitudinal bars are also located at the corners of the ties and thus cannot resist the horizontal component of the diagonal

Fig. 20 — Torsional failure of IW-series specimens

stress, it is compensated by the prestress. Therefore the ties can develop their full strength. Unfortunately no explanation can be offered for the discrepancy of the one exception in the TW series.

CONCLUSIONS

The results of this investigation regarding the behavior and strength of pretensioned concrete members under short-time torsional load may be summarized and conclusions may be drawn as follows:

1. St. Venant's elastic torsion theory is applicable up to failure of pretensioned concrete members since the inelasticity of the high strength concrete is not pronounced under short-time loading.

2. Of the various theories of failure, Rankine's overestimates while Coulomb's underestimates the torsional strength of prestressed concrete. The latter is in complete disagreement with experimental evidence. Cowan's theory overestimates the strength if the failure is predominantly cleavage type but conservatively predicts the strength if the failure is primarily fracture type.

3. The modification to Cowan's theory proposed here describes the failure of concrete as transitional from distinct cleavage to fracture. This modified theory closely predicts the torsional strength of concrete under prestress of not more than 0.60 of f_c' which is the range of stress encountered in most prestressed concrete construction. In essence, this theory is a close approximation of Mohr's generalized internal friction theory.

4. The Modified Cowan Theory depends on two independent properties of concrete; compressive strength and torsional strength. The latter is assumed to be equal to tensile strength and is given by an empirical relation $\tau_o = f_c' = 0.68\ (f_c')^{3/4}$. According to this theory the optimum prestress is 70 percent of the concrete compressive strength. With this prestressing effort the corresponding torsional strength of

the concrete is 2.81 times that of plain concrete if the concrete compressive strength is 6000 psi.

5. Ultimate moment of prestressed members of rectangular and T-sections without web reinforcement is the same as the cracking moment determined according to St. Venant's torsion theory and the modification of Cowan's criterion of failure.

6. Ultimate moment and cracking moment of prestressed members of I-section may be substantially different. The difference is greatly affected by the proportions of the section and the eccentricity of the prestressing force. However, the usable strength of such a section should be taken as the cracking moment, since the cracks due to torsion remain open on unloading and thus permanently damage the concrete member.

7. A concrete member without web reinforcement fails abruptly under torsion. This type of sudden failure can be avoided by using web steel as ties spaced not more than 0.4 of the depth of the member. This delayed failure was fully demonstrated by the specimens containing web steel of about 1.5 percent of the volume of the core enclosed within the ties.

8. The use of web reinforcement has no apparent effect on the elastic torsional behavior of a member.

9. Ultimate strength of prestressed members with web steel is equal to the sum of the cracking moment of the member and the moment resisted by the web reinforcement.

ACKNOWLEDGMENTS

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Title No. 57-59

Shell Construction — A New Approach

By WALTER E. RILEY

To reduce cost of forming thin shell concrete roofs, an economical method of construction is desired. The design, construction, and load testing of a 15 x 30 ft cylindrical test shell using a new approach is described.

■ SHELL ROOFS MAKE POSSIBLE unobstructed floor areas, permit wide latitude in structural layout, and enable the designer to fit many functional requirements. Moreover, shell roofs, and particularly the curved or arched types, offer major economies in the amounts of concrete and steel required in terms of area roofed. Unfortunately, these advantages often have been outweighed in the past by the high cost of the formwork required, for it has been necessary to build a form at roof level and to hold this form, usually from the ground, by a forest of supports and braces until the concrete has been placed and gained sufficient strength before removal. The large amount of material required for the formwork and the expense of the skilled labor required for erecting and dismantling the formwork have made the construction of concrete shell roofs in the United States costly, except in unusual instances.

It is apparent that reducing the formwork cost is highly desirable. The construction method for a cylindrical concrete shell described here appears to have merit. In this method a mound is formed, preferably of earth, at ground level on the building site. This mound is shaped to conform to the shape of the roof, being of round convex vertical section, if an arched roof is desired, or of inverted V-shaped vertical cross section when a hipped roof is wanted. After the mound is formed, the roof is cast over it and allowed to set. The set roof is then raised off the mound by hydraulic jacks or the like to its level in the structure and there supported permanently on columns.

A plurality of vertical sleeves is formed in the roof when the concrete is placed. Thus, forms or skeletons for the vertical sleeves are placed around the edges of the mound prior to placing the roof. The concrete is then cast around the sleeve forms at the same time it is placed over the mound, then the roof is raised on columns disposed in the sleeves. After the roof has been raised, the mound is removed or leveled off and floor and wall construction proceed in conventional fashion.

SHELL ANALYSIS

b = breadth of edge beam
E = modulus of elasticity
f = unit stress
H_L = horizontal line load applied along longitudinal edge
h = depth of edge beam
I = moment of inertia
L = length of shell between supports
M_ϕ = bending moment on radial face, considered positive when it produces tension in inner fibers
p = intensity of vertical load on a unit area
r = radius of shell measured to center line
S = tangential shearing force:
  S_L = tangential shearing live load applied along longitudinal edge
  S_b = shearing force applied at upper edge of beam
T = direct force component:
  T_ϕ = force component in transverse direction
  T_x = force component in longitudinal direction
t = thickness of shell
x = longitudinal distance measured from left support
Δ_v = vertical displacement
ϕ = angle measured from right edge of shell
ϕ_k = angle subtended by the edge of shell measured from the center line
V = vertical component of transverse force:
  V_L = vertical line load applied along longitudinal edge
  V_b = vertical load acting on edge beam
a, b, and c = coefficients

A test shell was built representing a half size model of a 30 x 60 ft roof. For the purpose of analysis, the dimensions were established as:

$b = 4$ in. $\quad r = 10.6$ ft
$h = 12$ in. $\quad t = 1\frac{1}{2}$ in.
$L = 30$ ft $\quad \phi_k = 45$ deg

Using the tables of ASCE *Manual* No. 31,* to three decimal places, the dead load calculated membrane values of T_ϕ and T_x at $x = L/2$ and $\phi = 0$; also S at $x = 0$ and $\phi = 0$ in lb per ft are:

$$T_\phi = \frac{4}{\pi} pr\,(-0.707) = -172 \qquad (1)$$

$$S = \frac{4}{\pi} pr \left(\frac{L}{r}\right)(-0.450) = -309 \qquad (2)$$

$$T_x = \frac{4}{\pi} pr \left(\frac{L}{r}\right)^2(-0.143) = -283 \qquad (3)$$

The line loads are:

$$V_L = -T_\phi \sin \phi_k = +121 \qquad (4)$$

$$H_L = -T_\phi \cos \phi_k = +121 \qquad (5)$$

$$S_L = +309 \qquad (6)$$

*"Design of Cylindrical Concrete Shell Roofs, *Manual* 31, American Society of Civil Engineers, New York, 1952, 177 pp.

ACI member **Walter E. Riley** is a licensed civil, structural, and mechanical engineer in Phoenix, Ariz. He has served as a consulting engineer on flat slab construction and thin shells and has authored a number of technical articles on methods of construction he has employed successfully.

These values are for the first term of the Fourier series.

Assuming no edge beam at bottom of shell, the line loads then result in:

$$T_x = \left[\frac{4}{\pi}\ pr\ (-0.143) + 1179\right]\left(\frac{L}{r}\right)^2 = 1214\left(\frac{L}{r}\right)^2 \quad (7)$$

$$\Delta_r = 3435\ \frac{L^4}{r^3\,t\,E} \quad (8)$$

The unit stress at edge of shell is $T_x/t = f_x$. Considering line loads V_b and S_b acting on shell by the edge beam:

$$f_x = [1214 - 15.44\,V_b - 1.13\,S_b]\left(\frac{L}{r}\right)^2 \frac{1}{t} \quad (9)$$

$$\Delta_r = [3435 - 71.08\,V_b - 1.79\,S_b]\left(\frac{L^4}{r^3\,t\,E}\right) \quad (10)$$

For the upper surface of edge beam the unit stress is:

$$f_x = \left[\ (-0.61\,V_b - 38.70)\frac{L}{h} + 1.27\,S_b\ \right]\frac{L}{b\,h} \quad (11)$$

$$\Delta_r = \left[\ (0.12\,V_b + 7.80)\frac{L}{h} - 0.19\,S_b\ \right]\frac{L^3}{b\,h^2\,E} \quad (12)$$

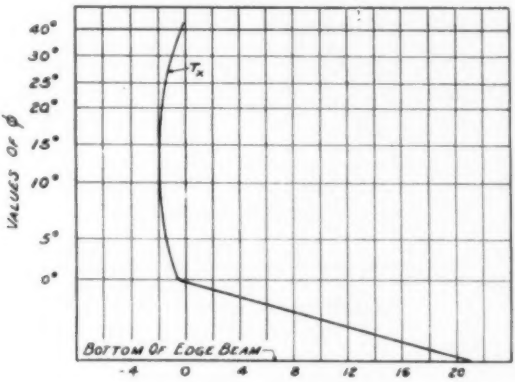


Fig. 1—Distribution of T_x, kip per ft

Equating and solving:

$$V_b = 9.8 \text{ lb per ft}; \quad S_b = 1003 \text{ lb per ft}$$

The true value of T_x using the corrected line loads is:

$$T_x = \left[\frac{4}{\pi} pr\,(-0.143) + 111.2\,(+15.44) - 121\,(-8.02) - 694\,(1.13)\right]\left(\frac{L}{r}\right)^2 = -603 \quad (13)$$

For other values of ϕ see Fig. 1.
At $x = L/2$ and $\phi = 0$,

$$T_\phi = \left[\frac{4}{\pi} pr\,(-0.707) + 111.2\,(0.707) + 121\,(0.707)\right] = \text{Negligible} \quad (14)$$

At $x = 0$, and $\phi = 0$,

$$S = \left[\frac{4}{\pi} pr\,(-0.450) - 694\,(0.350)\right]\left(\frac{L}{r}\right) = 996 \quad (15)$$

Since the analysis used assumes no torsional restraint at edge of shell, M_ϕ is zero at this point. For other values of ϕ,

$$M_\phi = (111.2\,a + 121\,b - 694\,c)\,r \quad (16)$$

Values of a, b, and c are taken from ASCE *Manual* No. 31. See Fig. 2 for values of T_ϕ, S, and M_ϕ.

Using Simpson's rule for determining the total value of the forces T_x around the shell and multiplying by their appropriate lever arm plus the total longitudinal force in the edge beam with its lever arm, the

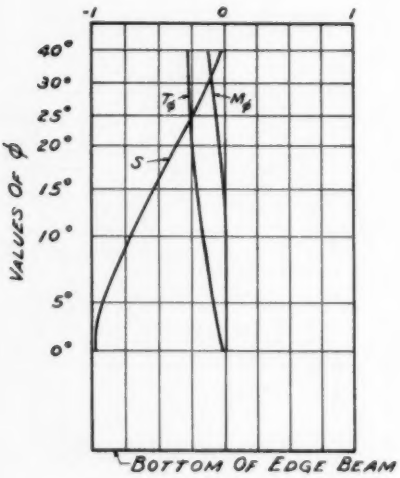


Fig. 2—Distribution of T_ϕ, kips per ft; S, kips per ft; and M_ϕ, ft-kips

value of the internal moment is 45.6 ft-kips. The external moment is:

$$\frac{W L^2}{\pi^2} = \frac{9.7^k (30)^2}{\pi^2} = 46.4 \text{ ft-kips} \quad \ldots\ldots (17)$$

This is considered close enough for design purposes. It should be kept in mind that W is a Fourier load, purely a mathematical convenience and is equal to:

$$\frac{2L}{\pi}\left(\frac{4}{\pi}\right)(\text{Shell weight} + \text{Edge beams weight}) \quad \ldots\ldots (18)$$

and is not the actual weight of roof.

Summing the vertical components of the variable tangential shear S at the end girder and adding the edge beam shear of:

$$(2L/\pi)\,(1/2)\,(4/\pi)\,(\text{edge beam weight per ft} + V_b) = 2.40^k$$

agreeing closely with $9.7^k\ (0.25) = 2.45^k$, giving an adequate shear check.

SHELL DESIGN

Shell reinforcing is shown in Fig. 3 and was designed for dead load plus 20 lb per sq ft live load. The diagonal shear steel at the corner was designed for 1.24 times the theoretical value of S to provide for omission of higher terms of the Fourier series. Additional reinforcing was added at the 2 ft square center opening.

It is becoming increasingly popular in the southwest United States to install air conditioning units on the roof of buildings using an air-

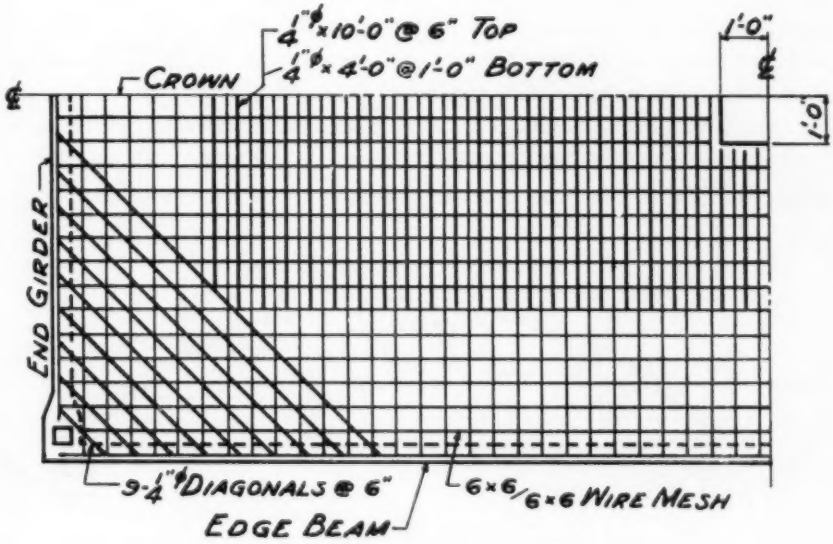


Fig. 3—Shell reinforcement

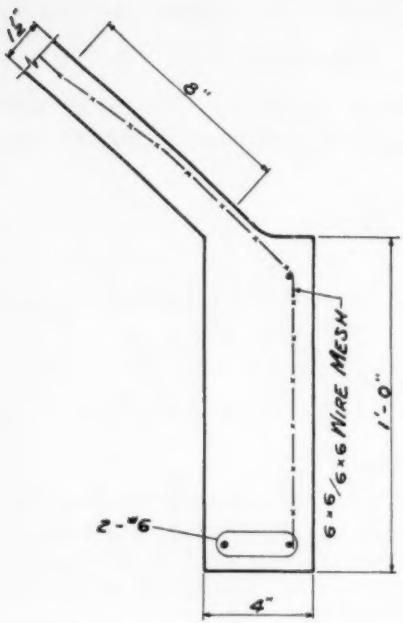


Fig. 4—Edge beam reinforcement

cooled compressor; thus an opening was made in the shell to simulate this condition. Edge beam reinforcing is shown in Fig. 4. Note that the mesh is raised at the shell junction to provide for edge disturbance even though ignored in analysis.

The loading of the end girders is the most uncertain part of the design. The usual determination of stress resulting from the vertical and horizontal components of the tangential shear S plus the weight of girder can be made without difficulty but a new set of loads must also be considered. At this point, one should refer to the specific method of construction. As shown in Fig. 5, a trench is dug below the original grade and a conventional wall footing is cast in the bottom of the trench. Next, the edge beam forms are placed on top of the footing and an earth mound is shaped appropriately in arch contour. The concrete shell is then cast integrally with the edge beam. After the roof has cured, it is raised with the edge beam which pulls up out of its form. Then, the grade beam is cast in the edge beam form and the earth mound is leveled off. In many instances the earth from the trench is sufficient to form the mound so that no external fill is required. End girders are cast at the same time in a similar manner

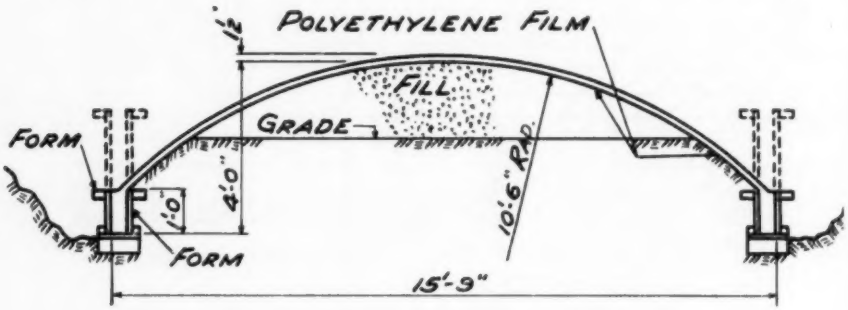


Fig. 5—Transverse section of shell in casting position

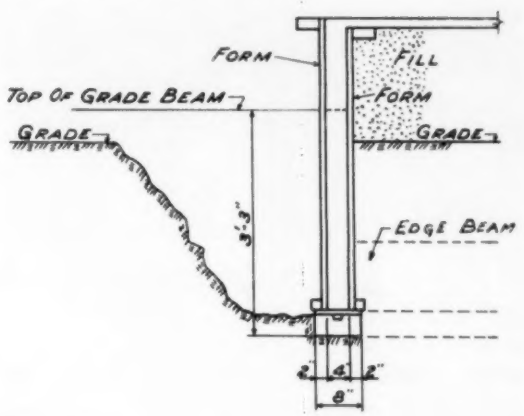
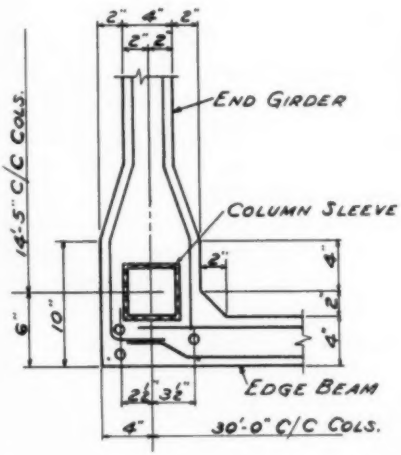


Fig. 6 — Section through end girder showing forming

as shown in Fig. 6. Column sleeves are placed at the corners (see Fig. 7). The column sleeve, shown in Fig. 8, is adapted to fit loosely around the column and is provided with an annular upper and lower flange with vertical guide pipes fastened to the outside.

Referring back to Fig. 6, it can be seen that consolidation of the fill behind the end girder form creates a lateral pressure. This pressure results in a friction force on the end girder which must be overcome at initial lifting. The magnitude of this force must be taken into ac-



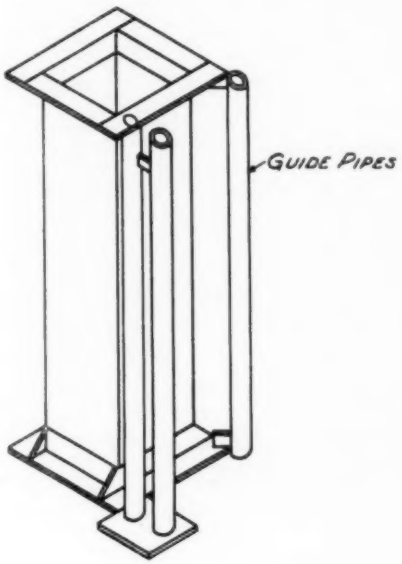


Fig. 7 (above)—Corner detail and column sleeve location. Fig. 8 (right)—Column sleeve

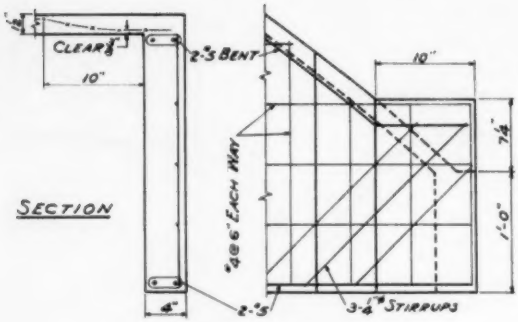


Fig. 9 — End girder reinforcement details

count in the design of the end girder. The cross section of the end girder is shown in Fig. 9. Mesh was lowered at the shell junction to provide for rim bending.

COLUMN FOOTINGS

Column footings are made large enough to accommodate the eccentric load of the jacking frame at lifting. Top of footings are slightly below top of grade beam footing to facilitate placing of lifting rods (see Fig. 10).

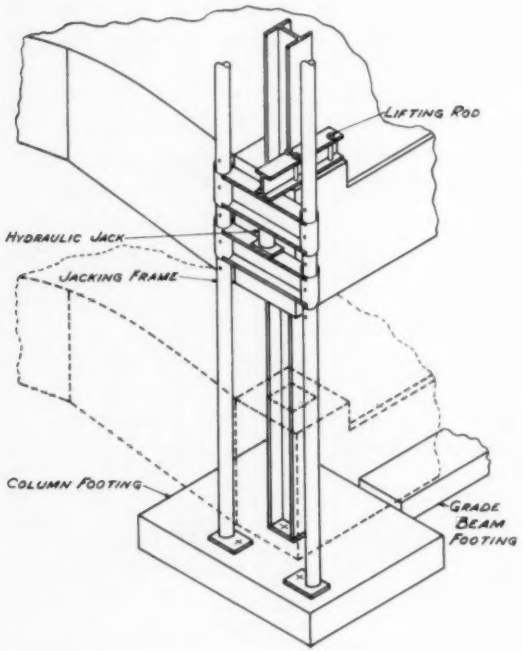


Fig. 10 — Lifting arrangement

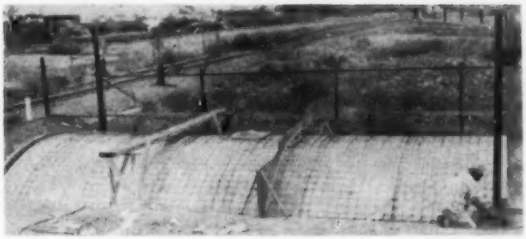


Fig. 11 — Shell with reinforcement in place prior to concreting. The screeding template can be seen in the center of the shell

ADDITIONAL ERECTION STRESSES

In addition to frictional forces exerted on the edge beams by the compacted earth, a moment is imposed on the ends of the edge beams due to the eccentricity of the jacking frame with respect to the center line of the end girder. This moment, however, is usually not serious.

TEST SHELL CONSTRUCTION

Fig. 11 shows the test shell with columns in place and ready for concrete. Maintaining the mesh in its proper position in the 1½-in. slab was a problem. Concrete pads were placed on the mound to support the mesh and were then cast into the shell. It was necessary in some cases to pull the mesh up where it had been stepped on between pads to get it into proper position. The earth mound was shaped by hand and compacted by hand roller. The wood screeding template can be seen and is cut to the outside radius of the shell.

After removing the outside edge beam forms and the outside end girder forms, the jacking operation was begun. A closeup of the lower portions of the jacking frame is given in Fig. 12. The shell is half raised in Fig. 13. Note that part of the polyethylene film has adhered to the shell.

No finishing was performed on the underside of the shell as the polyethylene film after removal left a smooth surface. Care, however, must be used in forming so as not to press the mesh into the earth

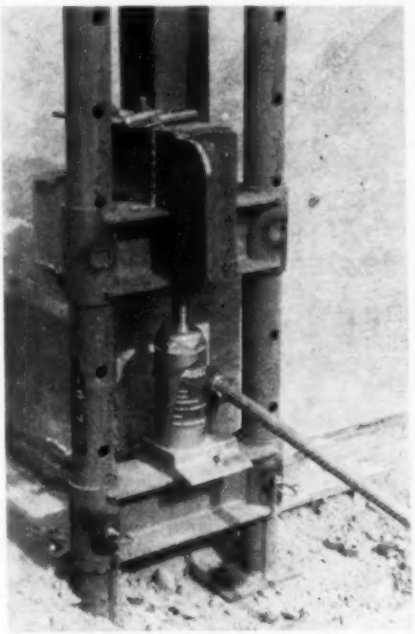


Fig. 12—Lifting jack

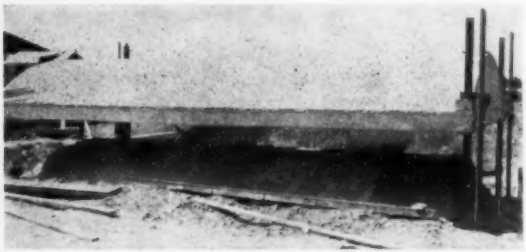


Fig. 13—Shell half raised. The polyethylene film can be seen clinging to the shell. This film made finishing of the underside unnecessary

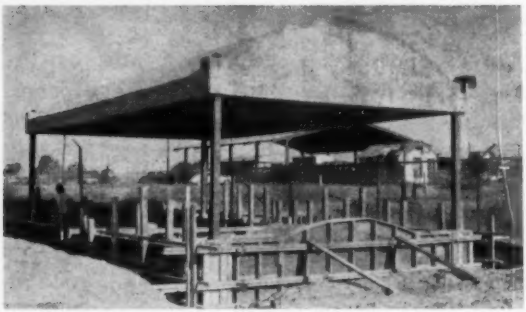


Fig. 14—Shell in final position. Grade beam forms in place. These forms are modified edge beam forms

mound as the resulting indentations are transferred to the underside of the shell and leave a pattern.

Fig. 14 shows the shell raised to full height. Using the original forms and modifying them slightly, the grade beams at ground level were formed as seen in Fig. 14.

LOAD TESTING

To check the validity of the theory assumed in the shell design, the structure was loaded with sacks of cement to give an average load of 40 lb per sq ft, twice that assumed in design. The sacks were confined to the central portion of the shell since they would slide off if placed near the edge beams. This created a different loading than the design assumed, but close enough for practical purposes. Fig. 15 shows the completed structure under load. Hair cracks developed at the center of the span at approximately 5 in. centers extending 3 to 4 in. up from the bottom of the edge beams. No cracks were discernible in the shell except the few shrinkage cracks present initially. The measured vertical deflection of the edge beams at the center of span was 0.63 in., comparing with the calculated deflection of 0.57 in.

Fig. 15—Completed structure with test loading in place

ROOFING

After completing the load test, the exterior of the roof was painted with three different types of waterproofing, silicone, acrylic polyvinyl, and polyvinyl. After 3 years, the roof was still in good condition and all three coatings were performing equally well.

CONCLUSION

Costs for this new method of construction compared to the usual method of forms and shores indicated a saving of over 50 cents per sq ft of shell. It has particular appeal for roofs of one and two bays where the usual forming cost is prohibitive since forms cannot be reused.

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Title No. 57-60

Fixed-End Moments in Columns of Asymmetrical Multispan Integral Frames due to Longitudinal Displacements

By SHU-T'IEN LI

Presents a general method for determining fixed-end moments in columns due to thermal changes, shrinkage, and rib-shortening of the monolithically continuous girder in asymmetrical integral bridge frames having different span lengths, column heights, and moments of inertia either constant or variable; and hence it is applicable to any combination of them in any form.

■ THE INCREASING USE OF multispan continuous integral frames in bridge structures of the interstate freeway system and the necessity of appropriate provision for thermal, shrinkage, and rib-shortening stresses induced therein, call for a general solution of column fixed-end moments which constitute an indispensable basic factor in the determination of final moments by the slope-deflection, moment-distribution, or relaxation method.

When the span lengths, column heights and their moments of inertia are symmetrical about the midpoint of the center span in the case of odd numbered spans, or about the center column(s) in even numbered spans, the determination of column fixed-end moments due to longitudinal displacements is relatively simple. But when all or some span lengths, column heights and their moments of inertia are asymmetrically different or variable, the determination of such column fixed-end moments must simultaneously satisfy all conditions of deformation, stiffness, and equilibrium.

Since this type of bridge structure seldom has more than five spans, the general analysis is developed although applications may be made to any composition from one to five spans with any column heights and stiffnesses by simply omitting nonexistent or substituting identical terms, or both. When the frictional resistances at end bearings are deleted, the equations hold true for pier-bents and building frames of similar composition.

DEVELOPMENT FOR TEMPERATURE RISE

Deformed structure and nomenclature

The deformed structure under temperature rise is diagrammatically shown in Fig. 1.

Let

l_1 *to* l_6 be the span lengths in ft

h_1 *to* h_4 be the column heights above top of footings, in ft

y_1 to y_4 be the vertical axial distances in ft along respective columns from the bottom

I_1 to I_4 be the moments of inertia at any section of respective columns, in ft⁴

Δ, Δ_c be, respectively, total and center-span expansion in ft, the former being typical in all cases and the latter typical in all odd numbered span cases

Δ_1 to Δ_4 be the longitudinal joint displacements in ft due to expansion resulting from temperature rise of the monolithically continuous girder with reference to the original vertical center lines of columns

H_1 to H_4 be the horizontal reactions in kips induced by thermal rise under fixed-end conditions of the columns

M_B, $M_{B'}$ to M_E, $M_{E'}$ be the fixed-end moments of the columns, in kip-ft

M_1 to M_4 be the moments in kip-ft at any section of respective columns

R_a, R_f be the dead load reactions in kips at the free-end supports

c_f be the coefficient of friction between contact surfaces at the free-end bearings; in case acceptable experimental result is unavailable for the particular metallic surfaces in contact, ¼ may be used

U_1 to U_2 be the internal strain energies of respective columns, in ft-kips

H_a, H_f be the frictional resistances at the free-end bearings, in kips

E_c be the modulus of elasticity of concrete in kips per sq ft

E_s be the modulus of elasticity of steel or steel reinforcement in kips per sq ft on the basis of 29,000,000 psi

n be the ratio of moduli of elasticity between steel and concrete; for lateral deflection of reinforced concrete columns designed for 3000-psi concrete, it may be taken at 8.

t be the temperature rise in deg F

c be the coefficient of expansion of concrete, which may be taken as 0.000006

Structural mechanics for columns of different constant *I*

If the integral substructures and the monolithically continuous girder had immense rigidity or stiffness as compared with the columns, both top and bottom of the latter would suffer no detectable rotation within the generally accepted accuracy of structural analysis. They would be virtually fixed, the tangents to the elastic curves thereat would remain vertical after the horizontal displacement due to thermal expansion of the monolithically continuous girder had taken place, the inflection points would be at midheight of each deflected column, and the fixed-end moments at top and bottom would be identical.

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Recognizing that fixed-end moments of constant-I columns due to a unit angle of lateral deflection are given by $6EI/h$ and angles of lateral deflection in radians by Δ/h, and neglecting the effect of any slight differential expansion of columns in the vertical direction, the fixed-end moments of the columns, their horizontal reactions, and the longitudinal joint displacements of the monolithically continuous girder would be related to each other by the following equations:

$$\left.\begin{aligned} M_B &= H_1\frac{h_1}{2} = \frac{6E_cI_1}{h_1^2}\,\Delta_1 \\ M_C &= H_2\frac{h_2}{2} = \frac{6E_cI_2}{h_2^2}\,\Delta_2 \\ M_D &= H_3\frac{h_3}{2} = \frac{6E_cI_3}{h_3^2}\,\Delta_3 \\ M_E &= H_4\frac{h_4}{2} = \frac{6E_cI_4}{h_4^2}\,\Delta_4 \end{aligned}\right\} \qquad (1)$$

These fixed-end moments are those with which the joints act on the columns. They are clockwise at B and C, and counterclockwise at D and E, producing tension on the outer fibers of columns. The fixed-end moment at each bottom is equal to that at each top, but with compression on the outer fibers.

In Eq. (1), if one of the M-, H-, and Δ-set of values are determined, the other two sets will become known. Hence, the system of simultaneous equations for solving the unknowns may be written in terms of either M's directly, or H's, or Δ's indirectly.

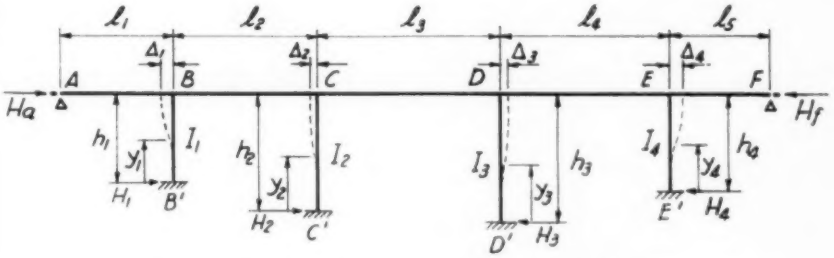

Fig. 1—Deformed structure under temperature rise

By definitions stated under nomenclature,

$$\Delta = ct(l_2 + l_3 + l_4) \quad \text{.....(2)}$$

hence the first deformation condition to be satisfied is

$$\Delta_1 + \Delta_4 = \Delta \quad \text{.....(A)}$$

The second deformation condition to be fulfilled is

$$\Delta_2 + \Delta_3 = c\,t\,l_3 = \Delta_c \quad \text{.....(B)}$$

And the third deformation condition to be met is

$$\frac{\Delta_1 - \Delta_3}{\Delta_4 - \Delta_3} = \frac{ctl_2}{ctl_4} = \frac{l_2}{l_4}$$

or

$$l_4\Delta_1 - l_4\Delta_2 + l_2\Delta_3 - l_2\Delta_4 = 0. \quad \text{.....(C)}$$

Again, according to the definitions given under nomenclature,

$$\left.\begin{aligned} H_a &= c_f R_a \\ H_f &= c_f R_f \end{aligned}\right\} \quad \text{.....(3)}$$

The condition of longitudinal equilibrium requires that

$$H_1 + H_2 - H_3 - H_4 + H_a - H_f = 0,$$

where, from Eq. (1),

$$H_1 + H_2 - H_3 - H_4 = 12E_c\left(\frac{I_1\Delta_1}{h_1^3} + \frac{I_2\Delta_2}{h_2^3} - \frac{I_3\Delta_3}{h_3^3} - \frac{I_4\Delta_4}{h_4^3}\right)$$

By calling

$$\left.\begin{aligned} \alpha_1 &= \frac{I_1}{h_1^3} \qquad \alpha_3 = \frac{I_3}{h_3^3} \\ \alpha_2 &= \frac{I_2}{h_2^3} \qquad \alpha_4 = \frac{I_4}{h_4^3} \end{aligned}\right\}$$

and

$$\left.\beta = \frac{H_f - H_a}{12E_c}\right\} \quad \text{.....(4)}$$

where

$$\left.E_c = \frac{E_s}{n},\right.$$

the equilibrium condition may be written in terms of Δ's thus:

$$\alpha_1\Delta_1 + \alpha_2\Delta_2 - \alpha_3\Delta_3 - \alpha_4\Delta_4 = \beta \quad \text{.....(D)}$$

The system of simultaneous Eq. (A), (B), (C), and (D) to be satisfied for the determination of longitudinal joint displacements may now be recapitulated in an array as follows:

Δ_1	Δ_2	Δ_3	Δ_4	C_i
1	0	0	1	Δ
0	1	1	0	Δ_c
l_1	$-l_1$	l_2	$-l_2$	0
α_1	α_2	$-\alpha_3$	$-\alpha_4$	β

(5)

Its solution by literal matrix inversion is not shorter in this case than by the determinantal method which shows that

$$\Delta_1 = \frac{\begin{vmatrix} \Delta & 0 & 0 & 1 \\ \Delta_c & 1 & 1 & 0 \\ 0 & -l_1 & l_2 & -l_2 \\ \beta & \alpha_2 & -\alpha_3 & -\alpha_4 \end{vmatrix}}{\begin{vmatrix} 1 & 0 & 0 & 1 \\ 0 & 1 & 1 & 0 \\ l_1 & -l_1 & l_2 & -l_2 \\ \alpha_1 & \alpha_2 & -\alpha_3 & -\alpha_4 \end{vmatrix}} = \frac{\Delta\left\{\begin{vmatrix} l_2 & -l_2 \\ -\alpha_3 & -\alpha_4 \end{vmatrix} - \begin{vmatrix} -l_1 & -l_2 \\ \alpha_2 & -\alpha_4 \end{vmatrix}\right\} - \Delta_c \begin{vmatrix} -l_1 & l_2 \\ \alpha_2 & -\alpha_3 \end{vmatrix} - \beta \begin{vmatrix} 1 & 1 \\ -l_1 & l_2 \end{vmatrix}}{\left\{-l_2(\alpha_2 + \alpha_3 + \alpha_4) - l_1\alpha_1\right\} + \begin{vmatrix} l_1 & l_2 \\ \alpha_1 & -\alpha_3 \end{vmatrix} - \begin{vmatrix} l_1 & -l_1 \\ \alpha_1 & \alpha_2 \end{vmatrix}}$$

whence

$$\Delta_1 = \frac{(l_2 + l_1)(\alpha_2\Delta_c + \alpha_4\Delta + \beta) + l_2(\Delta - \Delta_c)(\alpha_2 + \alpha_3)}{(l_2 + l_1)\,\Sigma\alpha}$$

Similarly,

$$\Delta_2 = \frac{(l_2 + l_1)(\alpha_3\Delta_c + \alpha_4\Delta + \beta) - l_2(\Delta - \Delta_c)(\alpha_1 + \alpha_4)}{(l_2 + l_1)\,\Sigma\alpha}$$

By putting

$$\left.\begin{aligned} K_1 &= (l_2 + l_1)(\alpha_3\Delta_c + \alpha_4\Delta + \beta) \\ K_2 &= l_2(\Delta - \Delta_c) \\ K_3 &= (l_2 + l_1)\,\Sigma\alpha \end{aligned}\right\} \quad \text{.....(6)}$$

we get

$$\left.\begin{aligned} \Delta_1 &= \frac{K_1 + K_2\,(a_2 + a_3)}{K_3} \\ \Delta_2 &= \frac{K_1 - K_3\,(a_1 + a_4)}{K_5} \end{aligned}\right\} \qquad (7)$$

and from (B),

$$\Delta_3 = \Delta_c - \Delta_2$$

from (A),

$$\Delta_4 = \Delta - \Delta_1$$

which will yield Δ_1 to Δ_4 for any composition of five spans.

In Eq. (6), as $\beta = (H_f - H_a)/12E_c$, where the difference in the numerator is usually small for any asymmetrical composition and equal to zero for symmetrical composition, and $12E_c$ is comparatively large numerically, β may be deleted from the equation under usual conditions, unless it affects the retained last significant figure for the desired accuracy.

Having determined the longitudinal joint displacements, the fixed-end moments at the top and bottom of the columns are given by Eq. (1).

Structural mechanics for columns of different variable *I*

In the most general case where the moments of inertia may be variable and vary differently along different columns, the variable I's may be expressed as functions of y's such that

$$I_1 = F(y_1), \quad I_2 = F(y_2), \quad \text{etc.} \qquad (8)$$

As by nomenclature, y's are measured from the bottom, the moments at any section and at the top of respective columns may be written as

$$\left.\begin{aligned} M_1 &= M_{B'} - H_1 y_1 \quad \text{and} \quad M_B = M_{B'} - H_1 h_1 \\ M_2 &= M_{C'} - H_2 y_2 \qquad\quad M_C = M_{C'} - H_2 h_2 \\ &\text{etc.} \qquad\qquad\quad \text{etc.} \end{aligned}\right\} \qquad (9)$$

from which it follows that the internal strain energies of the columns are given by

$$\left.\begin{aligned} U_1 &= \int_0^{h_1} \frac{(M_{B'} - H_1 y_1)^2}{2E_c I_1}\, dy_1 \\ U_2 &= \int_0^{h_2} \frac{(M_{C'} - H_2 y_2)^2}{2E_c I_2}\, dy_2 \end{aligned}\right\} \qquad (10)$$

etc.

As fixed-end moments at the bottom must satisfy the conditions

$$\frac{\partial U_1}{\partial M_{B'}} = 0, \quad \frac{\partial U_2}{\partial M_{C'}} = 0, \text{ etc.} \tag{11}$$

whence

$$\int_0^{h_1} \frac{M_{B'}}{I_1}\, dy_1 - H_1 \int_0^{h_1} \frac{y_1}{I_1}\, dy_1 = 0$$

giving the relation between $M_{B'}$ and H_1 as

$$M_{B'} = H_1 \int_0^{h_1} \frac{y_1}{I_1}\, dy_1 \Big/ \int_0^{h_1} \frac{dy_1}{I_1} \tag{12}$$

which reduces, for constant I_1, to $M_{B'} = H_1 h_1/2$, as it should.

Similarly, for $M_{C'}$, $M_{D'}$, and $M_{E'}$, the right-hand expressions will, respectively, have subscripts 2, 3, and 4.

To establish the relations between M_B and H_1, M_C and H_2, etc., the moments at any section of the columns are expressed in terms of M_B, M_C, etc., thus

$$\left.\begin{aligned} M_1 &= M_B + H_1 h_1 - H_1 y_1 \\ M_2 &= M_C + H_2 h_2 - H_2 y_2 \\ &\text{etc.} \end{aligned}\right\} \tag{13}$$

whence

$$\frac{\partial U_1}{\partial M_B} = 0 = \frac{1}{E_c} \int_0^{h_1} \frac{(M_B + H_1 h_1 - H_1 y_1)}{I_1}\, dy_1$$

giving

$$M_B = H_1 \frac{\displaystyle\int_0^{h_1} \frac{y_1}{I_1}\, dy_1 - h_1 \int_0^{h_1} \frac{dy_1}{I_1}}{\displaystyle\int_0^{h_1} \frac{dy_1}{I_1}} \tag{14}$$

which reduces, for constant I_1, to $M_B = -H_1 h_1/2$, as it should.

Similarly, for M_C, M_D, and M_E, the right-hand expressions will, respectively, have subscripts 2, 3, and 4.

Calling

$$\left.\begin{aligned} C_1 &= \left(\int_0^{h_1} \frac{y_1}{I_1}\,dy_1 - h_1 \int_0^{h_1} \frac{dy_1}{I_1}\right) \Big/ \int_0^{h_1} \frac{dy_1}{I_1} \\ C_2 &= \left(\int_0^{h_2} \frac{y_2}{I_2}\,dy_2 - h_2 \int_0^{h_2} \frac{dy_2}{I_2}\right) \Big/ \int_0^{h_2} \frac{dy_2}{I_2} \end{aligned}\right\} \quad (15)$$

etc.

we may write

$$\left.\begin{aligned} M_B &= H_1 C_1 \\ M_C &= H_2 C_2 \end{aligned}\right\} \quad (16)$$

etc.

The longitudinal joint displacement Δ_1 at B must be equal to the total moment about B of the differential elastic weights $M_1 dy_1/EI_1$ integrated over $B'B$, that is

$$\Delta_1 = \int_0^{h_1} \frac{M_1}{E_c I_1} (h_1 - y_1)\,dy_1$$

where

$$M_1 = H_1 (C_1 + h_1 - y_1),$$

giving

$$\Delta_1 = \frac{H_1}{E_c}\left[C_1 \int_0^{h_1} \frac{(h_1 - y_1)}{I_1}\,dy_1 + \int_0^{h_1} \frac{(h_1 - y_1)^2}{I_1}\,dy_1 \right] \quad (17)$$

In like manner, for Δ_2, Δ_3, Δ_4, the right-hand expressions will respectively have subscripts 2, 3, and 4.

Denoting

$$\left.\begin{aligned} C_1' &= \frac{1}{E_c}\left[C_1 \int_0^{h_1} \frac{(h_1 - y_1)}{I_1}\,dy_1 + \int_0^{h_1} \frac{(h_1 - y_1)^2}{I_1}\,dy_1 \right] \\ C_2' &= \frac{1}{E_c}\left[C_2 \int_0^{h_2} \frac{(h_2 - y_2)}{I_2}\,dy_2 + \int_0^{h_2} \frac{(h_2 - y_2)^2}{I_2}\,dy_2 \right] \end{aligned}\right\} \quad (18)$$

etc.

we have

$$\left.\begin{aligned}\Delta_1 &= H_1C_1'\\ \Delta_2 &= H_2C_2'\end{aligned}\right\} \qquad (19)$$

By combining Eq. (16) and (19), we obtain the counterparts of Eq. (1) for variable-I columns as

$$\left.\begin{aligned}M_B &= H_1C_1 = (C_1/C_1')\,\Delta_1\\ M_C &= H_2C_2 = (C_2/C_2')\,\Delta_2\\ M_D &= H_3C_3 = (C_3/C_3')\,\Delta_3\\ M_E &= H_4C_4 = (C_4/C_4')\,\Delta_4\end{aligned}\right\} \qquad (20)$$

The deformation and equilibrium conditions on which the system of simultaneous equations of unknowns are formed remain the same as for constant-I columns, and Eq. (A), (B), and (C) remain unchanged for variable-I columns, but Eq. (D) has to be rewritten. Now, Eq. (20) provides

$$H_1 + H_2 - H_3 - H_4 = \frac{\Delta_1}{C_1'} + \frac{\Delta_2}{C_2'} - \frac{\Delta_3}{C_3'} - \frac{\Delta_4}{C_4'}$$

By calling

$$\gamma_1 = \frac{1}{C_1'}, \quad \gamma_2 = \frac{1}{C_2'}, \text{ etc.} \qquad (21)$$

Eq. (D) becomes

$$\gamma_1\Delta_1 + \gamma_2\Delta_2 - \gamma_3\Delta_3 - \gamma_4\Delta_4 = \beta \qquad (D')$$

Thus, the system of simultaneous equations to be satisfied for the determination of longitudinal joint displacements resulting from thermal rise in the most general case of different variable-I columns takes the following array:

Δ_1	Δ_2	Δ_3	Δ_4	c_4
1	0	0	1	Δ
0	1	1	0	Δ_c
l_4	$-l_4$	l_2	$-l_2$	0
γ_1	γ_2	$-\gamma_3$	$-\gamma_4$	β

(22)

which is identical in form as Eq. (5) except for the magnitudes of the fourth row of coefficients. Therefore, the solution as represented in Eq. (6) and (7) for columns of different constant I will be equally operative for columns of different variable I by simply substituting γ's for α's in those equations.

For those who prefer the simplest matrix solution, reference is made to the author's paper on "Converging Matrix Algorisms for the Solution

of Non-homogeneous Simultaneous Algebraic Linear Equations," presented to the fifteenth national meeting of the Association for Computing Machinery, Aug. 22-26, 1960, at Marquette University, Milwaukee, Wis.

ADAPTATION TO THERMAL FALL AND SHRINKAGE

Thermal fall and shrinkage of the multispan integral frame induce fixed-end moments of proportional magnitudes on the columns, but of opposite sign to those caused by thermal rise. Denoting unit longitudinal and center-span displacements due to thermal fall and shrinkage by

$$-\partial_{fs} = -(ct_f + 0.0002) \quad (23)$$

and that due to thermal rise by

$$\partial_r = ct_r \quad (24)$$

we get a constant ratio

$$r = -\frac{\partial_{fs}}{\partial_r} = -\frac{ct_f + 0.0002}{ct_r} \quad (25)$$

When this constant ratio is multiplied by each column fixed-end moment due to thermal rise, the latter will automatically become corresponding fixed-end moment induced by thermal fall and shrinkage.

DEVELOPMENT FOR RIB-SHORTENING OR ELONGATION

In the case of rib-shortening or elongation of the monolithically continuous girder, if

N be the compression or tension in kips at any section of the girder
A be any transformed cross-sectional area in sq ft normal to the longitudinal axis of the girder
dl be any differential length in ft along the longitudinal girder
Δ' be the shortening or elongation in ft of any span
k be any span-number subscript 2, 3, or 4

then

$$\Delta_k' = \pm \int_0^{l_k} \frac{N_k}{A_k E_c}\, dl_k$$

whence

$$\left.\begin{aligned} \Delta_c = \Delta_3' &= \pm \int_0^{l_3} \frac{N_3}{A_3 E_c}\, dl_3 \\ \text{and} \\ \Delta = \Sigma\Delta_k' &= \sum_{k=2}^{4} \pm \int_0^{l_k} \frac{N_k}{A_k E_c}\, dl_k \end{aligned}\right\} \quad (26)$$

where the positive sign denotes elongation and the negative sign shortening.

The analogous sets of simultaneous equations for the unknown longitudinal joint displacements may now be written by substituting

$$\frac{\Delta_1 - \Delta_2}{\Delta_4 - \Delta_3} = \frac{\Delta_2'}{\Delta_4'}$$

or

$$\Delta_4' \Delta_1 - \Delta_4' \Delta_2 + \Delta_2' \Delta_3 - \Delta_2' \Delta_4 = 0 \qquad \text{(C')}$$

for Eq. (C) in both cases of (a) constant-I and (b) variable-I columns. Thus, we have the following arrays:

(a)

Δ_1	Δ_2	Δ_3	Δ_4	c_i
1	0	0	1	Δ
0	1	1	0	Δ_c
Δ_4'	$-\Delta_4'$	Δ_2'	$-\Delta_2'$	0
α_1	α_2	$-\alpha_3$	$-\alpha_4$	β

(27)

(b)

Δ_1	Δ_2	Δ_3	Δ_4	c_i
1	0	0	1	Δ
0	1	1	0	Δ_c
Δ_4'	$-\Delta_4'$	Δ_2'	$-\Delta_2'$	0
γ_1	γ_2	$-\gamma_3$	$-\gamma_4$	β

(28)

whose solution is analogously given by Eq. (6) and (7) by simply substituting appropriate elements in the above equations for corresponding ones in Eq. (5).

GENERAL APPLICABILITY TO DIFFERENT COMPOSITIONS

Application to symmetrical five-span bridge frame or three-span pier bent (Case 1)

In this case, by virtue of symmetry, $l_2 = l_4$, $h_1 = h_4$, $h_2 = h_3$, $I_1 = I_4$, $I_2 = I_3$, $\alpha_1 = \alpha_4$, $\alpha_2 = \alpha_3$, and $H_f - H_c = 0$.
Therefore

$$\left.\begin{aligned} \Delta_1 = \Delta_4 = \frac{\Delta}{2} &= \tfrac{1}{2}\, ct\,(2l_2 + l_3) \\ \Delta_2 = \Delta_3 = \frac{\Delta_c}{2} &= \tfrac{1}{2}\, ctl_3 \end{aligned}\right\} \qquad (29)$$

These will be identical with the results that would be obtained from Eq. (6) and (7) by substituting therein the above symmetrically located equal factors.

By Eq. (6), $K_1 = 2l_2\ (\alpha_2\,\Delta_c + \alpha_1\,\Delta)$, $K_2 = l_2\ (\Delta - \Delta_c)$, and $K_3 = 4l_2\ (\alpha_1 + \alpha_2)$. When these are substituted into Eq. (7), we get $\Delta_1 = \Delta_4 = \Delta/2$ and $\Delta_2 = \Delta_3 = \Delta_c/2$ which agree with those given by Eq. (29), *quod erat demonstrandum*.

Application to asymmetrical four-span bridge frame or two-span pier bent (Case 2)

By deleting the terms l_4, h_4, I_4, α_4, and Δ_4, Eq. (6) gives $K_1 = l_2(\alpha_3\,\Delta_c + \beta)$, $K_2 = l_2(\Delta - \Delta_c)$, $K_3 = l_2(\alpha_1 + \alpha_2 + \alpha_3)$, where β may be put $= 0$; and hence Eq. (7) yields the desired results as

$$\left.\begin{aligned} \Delta_1 &= \frac{\Delta(\alpha_2 + \alpha_3) - \Delta_c\alpha_2}{\alpha_1 + \alpha_2 + \alpha_3} \\ \Delta_2 &= \frac{\Delta_c\,(\alpha_1 + \alpha_3) - \Delta\alpha_1}{\alpha_1 + \alpha_2 + \alpha_3} \\ \Delta_3 &= \Delta_c - \Delta_2 \end{aligned}\right\} \qquad (30)$$

Application to symmetrical four-span bridge frame or two-span pier bent (Case 3)

Besides deleting terms as stated under Case 2, put $l = l_2 = l_3$, $h_1 = h_3$, $I_1 = I_3$, $\alpha_1 = \alpha_3$, $\alpha_2 = 0 = \Delta_2$, $\Delta_1 = \Delta_3$. The premises dictate

$$\Delta_1 = \Delta_3 = \frac{\Delta}{2} = ctl \qquad (31)$$

But the first line of Eq. (30) per se will hold and yield $\Delta_1 = \Delta_3 = \Delta\alpha_1/2\alpha_1$ or $\Delta\alpha_3/2\alpha_3 = \Delta/2$ being identical with Eq. (31), *quod erat demonstrandum*.

Application to asymmetrical three-span bridge frame or one-span pier bent (Case 4)

Besides the terms deleted under Case 2, further omit the terms α_3, and put $\Delta_c = 0$. Thus, the first line of Eq. (30) becomes

$$\Delta_1 = \frac{\Delta\alpha_2}{\alpha_1 + \alpha_2} \text{ (in direction shown in Fig. 1)}$$

and the second line of Eq. (30) becomes

$$\Delta_2 = \frac{-\Delta\alpha_1}{\alpha_1 + \alpha_2} \text{ (in opposite direction shown in Fig. 1)}$$

The minus sign for Δ_2 indicates the actual longitudinal displacement Δ_2 for Case 4 should be to the right side of C in Fig. 1, while it is shown there to its left for the general multispan case. It further demonstrates that correct mathematical relations not only give "infallible magnitudes" but also "infallible senses."

A plus sign for Δ_2 would have been automatically obtained by using Spans 1, 3, and 5 for the three-span bridge frame and Span 3 for the one-span pier bent; i.e., putting in Fig. 1, $l_2 = l_4 = 0 = \Delta_2 = \Delta_3$; omitting

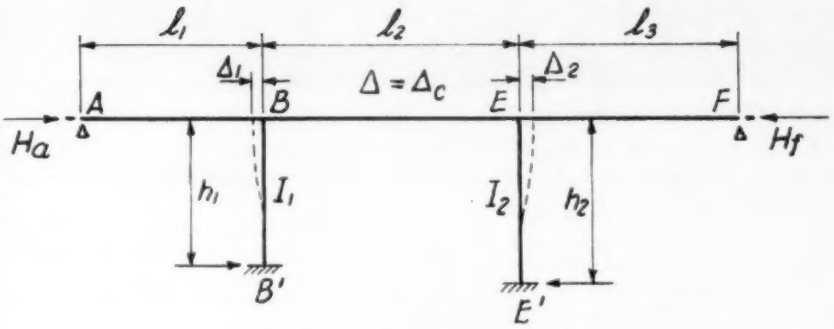

Fig. 2—Asymmetrical three-span frame

Columns 2 and 3; redesignating Δ_4, I_4, h_4, H_4, respectively, as Δ_2, I_2, h_2, H_2; l_3 as l_2, and l_2 as l_3; then Fig. 1 becomes Fig. 2, and $\Delta = \Delta_c$. It is to be noted that in this case the typical total expansion Δ and the typical center-span expansion Δ_c unify into an identical quantity. By assuming $H_f - H_a = 0$, Eq. (6) gives $K_1 = l_2\alpha_2\Delta$, $K_2 = 0$, $K_3 = l_2\Sigma\alpha = l_2(\alpha_1 + \alpha_2)$; and Eq. (7) yields

$$\left.\begin{aligned} \Delta_1 &= \frac{\Delta\alpha_2}{\alpha_1 + \alpha_2} \\ \Delta_2 &= \frac{\Delta\alpha_1}{\alpha_1 + \alpha_2} \end{aligned}\right\} \quad \text{(32)}$$

which are the same in magnitudes as before, but both in the same directions as shown in Fig. 2.

Application to symmetrical three-span bridge frame or one-span pier bent (Case 5)

For this case, simply put $\alpha_1 = \alpha_2$ in Eq. (32). Thus,

$$\Delta_1 = \Delta_2 = \frac{\Delta}{2} \quad \text{(33)}$$

CONCLUSIONS

In closing, the author expresses these concluding statements:

1. This paper has uniquely derived fixed-end moments in columns due to longitudinal-girder displacements of asymmetrical multispan integral frames.

2. The formulas as presented are of general applicability to the determination of fixed-end moments due to thermal rise or fall, shrinkage, and rib-shortening or elongation, of the longitudinal girder.

3. They are applicable to integral frames of bridges, pier bents, and buildings, in any composition not more than five spans or three bays,

asymmetrical or symmetrical, with either variable or constant moments of inertia.

4. The method may be extended to cover more spans or bays when such cases arise.

5. All numbered formulas are presented in proper sequence for any form of systematic numerical analysis including programed electronic computation.

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Blombach viaduct (Die Blombach Talbrücke)

E. BAUMANN, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 8, Aug. 1960, pp. 352-358

Reviewed by FERDINAND S. ROSTASY

The Blombach bridge near Wuppertal, Germany, represents, with its 990 ft clear span and 1090 ft total length, one of the largest reinforced concrete structures built after the war. The structure is a classical unsymmetrical fixed-end arch. Detailed information is given as to the construction of the foundation and formwork. Design, reinforcement, and concreting are discussed.

Construction Techniques

Cut form cost to 7¢ per sq ft

JOE TODARO, *Construction Equipment*, V. 22, No. 3, Sept. 1960, pp. 35-41

Reviewed by MARY K. HURD

Combination form-and-shore system used on the 11-story, 140,000 sq ft addition to Charlotte Memorial Hospital, Charlotte, N.C., cost 7 cents per sq ft for erecting and stripping combined. Almost entirely slab work, the job progressed at one 13,000 sq ft floor per week. Carefully planned erection scheme was based on using only six prefabricated components—one shore, one beam, four panels—per 48 sq ft of shored slab. The adjustable steel shore with four holes in its head plate provided snap-on lock for the pins in lower flange of 6 x 4-in. support beam. Two identical pins in the upper flange fitted snugly in corresponding holes of steel framed plywood form panels. Scalloped outside edge of slab, uniform from floor to floor, was formed with specially fabricated metal panels, and custom-made corner pieces were also used on drop beams around elevator openings.

Expanded slate aggregate was used for the lightweight 4000-psi mix, which was delivered by truck mixers and placed by an electric tower crane with two 1½ cu yd buckets in service. Tower crane also speeded handling of other construction materials, including precast, exposed aggregate facing panels weighing from 1500 to 4000 lb each.

Ganged forms shape seawall

JOHN REHFIELD, *Construction Equipment*, V. 22, No. 3, Sept. 1960, pp. 47-51

Reviewed by MARY K. HURD

Rented factory-built prefabricated forms were ganged for crane handling on a 2149 ft long wharf at the Mayport, Fla., Naval Station. Wharf straightens ragged shore line of an inlet, has about one-quarter of its length in open water. Form ganging speeded erection and alignment, saved carpentry work in the water.

Concrete wall encases the upper 15 ft of the wharf's sheetpile backbone, so reinforcing had to be set around piling, and form ties were threaded through holes in the piling. Ganged units measured 15 x 22½ ft. Strong-

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backs of 2 x 12 lumber projecting above wall form braced the bottom of the gang and eliminated underwater tie-rod setting. Timber grillage behind panels prevented warping during lift, eliminated need for external bracing during placement.

Two tower cranes handle materials for apartments

Contractors and Engineers, V. 57, No. 12, Dec. 1960, pp. 88-91

Well-illustrated description of hoisting operations of two all-electric tower cranes during construction of the 16-story Comstock Apartments, San Francisco. The two 100-ft booms work two floors from a setting; while one brings concrete from transit mixer to forms, the other handles general lifting jobs. The remote control of the rig is not only quiet but adds safety and efficiency to the job.

Concrete by grout injection (in Dutch)

J. F. Brants, *Cement* (Amsterdam), V. 12, No. 8, Apr. 1960, pp. 680-684

Reviewed by John W. T. Van Erp

Describes prepacked concrete according to the Colcrete process. Preplaced aggregate is injected with a colloidal cement-sand mixture, including a dispersion agent. The application of this method to such items as mass-concrete, foundations, harbor construction, and heavy radiation shielding concrete structures is mentioned. Experience with road construction in England and Germany is described and also jobs with intricate embedded elements, conduits, and reinforcement.

Construction of industrial buildings with precast reinforced concrete units (Bauen mit Stahlbetonfertigteilen—Hallen, Industriebauten)

Laszlo Mokk, Publishing House of the Hungarian Academy of Sciences, Budapest, 1960, 473 pp.

Reviewed by Mary K. Hurd

This comprehensive work was first published in Hungary in 1955, subsequently was translated for several eastern European countries, and now appears in a German-language edition. A profusion of good illustrations will make it interesting even to those whose knowledge of German is scant.

Noting that precasting is most advantageous in areas where there is a relative scarcity of timber and steel, plus a need to economize on labor, the author cites statistics on recent production of precast units in Europe. He also compares advantages of factory precasting with precasting at the job site, and discusses precasting in general versus cast-in-place construction.

Large precast members must be designed for lifting stresses as well as for their role in the completed structure, and a lengthy second chapter deals with problems of structural analysis. Cross-section and elevation drawings are presented for numerous typical members, and different methods of joining the members are well detailed.

Production of the units is dealt with in a third chapter, with suggestions on layout for precasting, both at the job site and in the plant, including the lifting of large members cast at the site. Considerable attention is given to forms and arrangement of casting beds, placing of concrete, attaining special surface effects, stripping the forms, and storage of the completed units.

Eight different classes of lifting devices are described, including light cranes, tower cranes, masts, derricks, etc., and all are pictured in detail. Hoisting techniques for various types of precast elements are covered, and accompanying action photographs are impressive. Systems for erection and assembly of beams, frames, arches, walls, and roofs are explained, with considerable details such as attention to lifting points, arrangement of cables, and sequence of crane operations.

Noting that much is to be learned from successfully completed precast construction jobs, the author presents a 170-page section on outstanding examples of contemporary precast construction. About 70 structures are described, many with cross-sectional detail drawings and views during erection

as well as of the completed structure. Majority of the examples are European, with a few from Canada, the United States, and Algeria. In the category of "special structures" are described lamella roofs, cooling towers with the form of hyperboloid of rotation, cement storage silos, and Nervi's small sports arena for the Olympic Games in Rome.

A brief concluding section compares construction costs for precast concrete, cast-in-place concrete, and steel structures.

Dams

Concrete "dowels" anchor cliff

A. E. Niederhoff, *Engineering News-Record*, V. 165, No. 16, Oct. 20, 1960, pp. 45-46, 48

Emphasizes construction difficulties of the 280 ft high concrete arch dam at the Kikuan hydroelectric project, Formosa. Because of torrential rains and poor rock foundation the dam was stabilized with 5 ft thick reinforced concrete "pins."

Design of buttress dams of triangular profile and constant web thickness

Janusz Ryszard Rydzewski, *Proceedings*, Institution of Civil Engineers (London), V. 16 (Session 1959-60), July 1960, pp. 271-282
Reviewed by Aron L. Mirsky

Based on the usual (British) design criteria and classical theory of stress distribution, tables and charts suitable for preliminary design are developed, as determined by either maximum allowable stresses or economy (taken as minimum volume).

Mareeba-Dimbulah irrigation project—Design and construction of Tinaroo Falls Dam

N. J. Butler and F. M. Learmonth, *Journal*, Institution of Engineers, Australia (Sydney), V. 32, No. 10-11, Oct.-Nov. 1960, pp. 249-263

Describes the civil engineering investigations, design, and construction of Tinaroo Falls Dam in North Queensland, Australia. Principal feature of the dam are: maximum height of dam from stream bed, 136 ft; maximum height of dam from deepest foundation level, 142 ft; length of dam, 1750 ft; concrete in dam, 292,000 cu yd.

Design

Design of flat slabs without dropped panels (in Swedish)

Arne Lindh, *Nordisk Betong* (Stockholm), V. 4, No. 3, 1960, pp. 189-208
Reviewed by Margaret Corbin

Presents a method of flat slab design in which simplified equations for the calculation of moments and shearing forces have been deduced as closely in accordance with the theory of elasticity as possible, without involving any excessively intricate calculation procedure. Torsional moments have also been taken into account, and it has been demonstrated that this can be done solely by correcting the length of members. This method has been devised for flat slabs without drop panels, but can likewise be applied to flat slabs with drop panels.

Annular and spherical shells over rectangular plan (in German)

E. Dulacska, *Acta Technica Academiae Scientiarum Hungaricae* (Budapest), V. 26, No. 3-4, 1959, pp. 349-356
Hungarian Technical Abstracts V. 12, No. 3, 1960

The development of construction technology demands that prefabrication keep in step in the field of shell structures as well. Economical prefabrication of dome shells can be realized only if the shell surface can be assembled of identical elements. Similar surfaces are referred to as annular, they can be divided into identical sections. The sphere is a special case of the annulus. The determination of membrane forces in this type of shell is dealt with. An even polynomial which accurately satisfies boundary conditions is adopted as the stress function. The differential equation of the shell is satisfied accurately only at designated points, while at others merely approximately. The method is illustrated by a numerical example which proves that

the quadratic mean error due to the approximating character of the method is 2.95 percent, the method thus being sufficiently accurate for practical purposes.

Design of biaxially loaded columns by ultimate load methods

F. N. PANNELL, *Magazine of Concrete Research* (London), V. 12, No. 35, July 1960, pp. 99-106
AUTHOR'S SUMMARY

Describes a method of defining the curves relating ultimate load and moment for rectangular columns having an equal area of steel in each face, and subjected to any radial eccentricity of loading. This leads to a method of accurate and rapid design.

The validity of the approach is demonstrated by case analysis.

Composite designs in precast and cast-in-place concrete

ARTHUR R. ANDERSON, *Progressive Architecture*, V. 41, No. 9, Sept. 1960, pp. 172-176

Briefly discusses considerations for composite construction and describes and details tests of connections. Also describes and pictures joint details for column-to-footing, wall-to-column, beam-to-column, and beam-to-slab joints. Many pictures illustrate practical applications of these joint details on actual construction jobs.

Control of stresses in concrete arched structures and arches by means of one-sided or unsymmetrical reinforcement (self-stressed arches, etc.) (in Russian)

YA S. FAIN, *Tr. Saratovsk. Avtomob.-dor. In-ta*, V. 15, No. 1, 1957, pp. 52-70; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 2, 1959, Rev. 1978
APPLIED MECHANICS REVIEWS
V. 13, No. 10, Oct. 1960

A method is proposed for the self-stressing of concrete arches which results in the decrease of the magnitude of the tension stresses in the arch. Calculations are worked out which take into account the concrete's creep, reducing the amount of metal used. In his investigations on the critical stresses in the ferro-concrete components the author starts with the assumptions that the cohesion between the reinforcement and the concrete is not broken; that the modulus of elasticity of the concrete becomes a constant in the course of time; that the concrete and the steel conform to Hooke's principle; that the concrete is in the first stage of the stressed state (up to the appearance of cracks indicating disruption of the concrete); and that the distribution of stresses with height in the section is rectilinear. The method is extended to cover cases of complete arches as well.

Bending moments in statically determinate reinforced concrete trusses at ultimate load

VALERIU PETCU, *The Indian Concrete Journal*, (Bombay), V. 34, No. 9, Sept. 1960, pp. 347-349

Presents a method to determine the bending moments in reinforced concrete trusses. Results were compared with those based on ordinary elastic analysis. Concludes that at the ultimate load the bending moments reduce the carrying capacity of reinforced concrete trusses more than the bending moments obtained by the elastic analysis.

Simplified analysis of circular arches (Calcul simplifie des arcs circulaires)

R. MASSE, *Le Génie Civil* (Paris), V. 136, No. 21, Nov. 1, 1959, pp. 438-441
Reviewed by ARON L. MIRSKY

Studied here are many cases of practical interest, using method developed by author in previous papers:* (1) circular pipe under two diametrically opposed forces; (2) circular pipe, under its own weight; (3) circular pipe, completely filled with liquid; (4) circular pipe, with vertical load at crown and vertical reactions acting at ends of the horizontal diameter; and (5) pipe in cradles.

*_Le Genie Civil_, V. 134, No. 20, Oct. 15, 1957, pp. 423-424; V. 135, No. 3, Feb. 1, 1958, pp 64-66; V. 136, No. 10, May 15, 1959, pp. 222-223; "Current Reviews," ACI JOURNAL, V. 32, No. 4, Oct. 1960 (*Proceedings*, V. 57) p. 471.

Two-span symmetrical portal frames under eccentric loads (Portiques doubles symetriques sollicites par l'action d'efforts excentres)

R. Masse, *Le Génie Civil* (Paris), V. 137, No. 3, Feb. 1, 1960, pp. 61-62
Reviewed by Aron L. Mirsky

Symmetry is used to derive simplified expressions for evaluating the redundants in two-span frames with horizontal or sloping cross-beams. A numerical example is included.

Reaction influence lines for continuous deep beams

F. E. Archer and E. M. Kitchen, *Civil Engineering Transactions* (Sydney), V. CE2, No. 2, Sept. 1960, pp. 77-84
Authors' Summary

The influence lines for reaction in two-span deep beams, of three ratios of depth to span and two ratios of width of supports to span, have been determined experimentally by a photoelastic technique. An approximate analytical method applicable to all continuous deep beams loaded on the upper edge has been developed. In addition, an example of an application of the method to the design of a continuous reinforced concrete deep beam is given.

Analysis of polygon-shaped slabs (Berechnung der Platten in Polygonalform)

Zlatko Modob, *Der Bauingenieur* (Berlin), V. 34, No. 11, Nov. 1959, pp. 427-437
Reviewed by Aron L. Mirsky

Starting with the general case of the irregular hexagon, with edges free or fully encastré, author then studies the triangular slab with three unequal encastré sides and constant thickness under uniformly distributed load, the results for which are given in the form of three-dimensional plots of deflections, moments, twisting moments, transverse forces and boundary pressures. Finally, charts are given for fixed-end moments for isosceles-triangular slabs under (1) uniformly distributed load, (2) load varying linearly along the altitude, (values for other loadings, e.g., trapezoidal, can be obtained by superposition). These charts, based on calculated values for four values of the apical angle (45, 60, 80, and 120 deg) are intended for design use.

Articulated continuous beam

Reginald G. Robertson, *Proceedings*, Institution of Civil Engineers (London), V. 15, Session 1959-60, Feb. 1960, pp. 105-118
Reviewed by Aron L. Mirsky

Author's previous study of three-span beams [*Proceedings* ICE V. 13, Aug. 1959, pp. 467-476; "Current Reviews," ACI Journal, V. 32, No. 1, July 1960, (*Proceedings*, V. 57, p. 114] is extended to five-span articulated beams and to beams with an infinite number of spans either side of the central "drop-in" section. Methods and charts are developed and presented for finding the correct span proportions, depths, and (where necessary) haunch sizes in both reinforced and prestressed concrete; a five-span beam in both materials is used as a worked example.

Determination of the stresses and forces in a ferro-concrete slab joined to a steel beam due to the reaction of the concrete and to the temperature (in Russian)

V. N. Mastachenko, *Trudi Moskovskogo In-ta Inzh. Zh.-d. Transp.*, No. 101, 1958, pp. 81-114; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 6, 1959, Rev. 6867
Applied Mechanics Reviews V. 13, No. 12, Dec. 1960

Results are given for the calculations of the problems given in the title, the method used being that of the plane problem in the theory of elasticity, while creep is taken into consideration where the shrinkage of the concrete is concerned. The calculations are carried out by a procedure previously developed by the author [*Trudi Moskovskogo In-ta Inzh. Zh.-dor. Transp.* 1957, (91), pp. 184-220]. Methods are indicated which improve the convergence of the series. Comparing the results of the calculations by the proposed procedure with those obtained by methods used by other investigators,

the author comes to the conclusion that the latter give somewhat high values for the corresponding calculated magnitudes.

Foundation vibrations

F. E. Richart, Jr., *Proceedings*, ASCE, V. 86, SM4, Aug. 1960 (Reprinted as *Technical Paper* No. 192, Florida Engineering and Industrial Experiment Station, Gainesville, 1960, 34 pp.)

Theoretical solutions for the vibrations of foundations resting on soil have been studied and evaluated. For vertical vibrations of a machine foundation, it is usually necessary to use the theory for an oscillator resting on an elastic semi-infinite body because of the important effects introduced by oscillations of the soil. This effect becomes of less importance if the force of the oscillator varies with the frequency of rotation. The use of Sung's theory for vertical foundation vibrations is outlined and curves are presented which may be used for analysis or design.

By comparison of the ordinary theory of vibration, which considers the spring to be weightless, to the theory for an oscillator on semi-infinite elastic body, it is shown that the ordinary theory is usually satisfactory for determining the natural frequency for the rocking and sliding modes of vibration of the foundation.

To evaluate the dynamic characteristics of a foundation, it was found necessary to determine the shear modulus and Poisson's ratio of the underlying soil. Typical values of these soil constants are given and methods for establishing them at a particular site are described. The shear modulus of a granular soil should increase approximately with the 1/3 power of the confining pressure.

Two examples are included to show the details of application of the theoretical methods. A diagram is also included which illustrates the possibility of stiffening the foundation, by the use of point-bearing piles to rock, to increase the natural frequency of vertical vibration.

Analysis of prismatic folded structures (in Russian)

Kh. Kh. Laull, *Trudi Tallinsk. Politekhn. In-ta* (*A*), No. 56, 1954, pp. 1-28; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 8, 1958, Rev. 9261

Applied Mechanics Reviews V. 13, No. 10, Oct. 1960

Examples are given of the calculation of a folded roof, a theatre balcony, and a reinforced concrete bunker, to illustrate the application of the suggested method of calculating prismatic shells (called the method of approximated shear stresses). Critical consideration is given to the method recently proposed by Craemer [H. Craemer, ACI Journal, V. 24, No. 8, Feb. 1953 (*Proceedings*, V. 49), p. 549] for the analysis of prismatic folded structures, and the method of Kazinczy (G. Kazinczy, Intern. Verein. für Brueckenbau und Hochbau, 3rd Congress, Liege, 1948) for the calculation of folded structures in the presence of cracks.

The theory of a new funicular shallow shell of double curvature

G. S. Ramaswamy, *The Indian Concrete Journal* (Bombay), V. 34, No. 9, Sept. 1960, pp. 336-341, 367-368

Author's Summary

In the design of shells of double curvature for roofs, the usual procedure is to select the shape of the shell first and carry out a stress analysis later. This illogical method often leads to lengthy and tedious calculations. In this paper, the process is reversed. A desired state of stress — compression throughout the shell — is assumed and the corresponding shape of the surface is derived. It is thus possible to do away with stress analysis altogether. This approach has resulted in the development of a new shallow shell of double curvature which may be described as a funicular shell. The shape of such shells to suit any given boundary geometry can be found with comparative ease using analytical or experimental methods. Equations to the surface of such a shell over rectangular, square, elliptical, circular, and triangular ground plans, being the most common, are discussed in detail. Com-

prehensive tables for setting them out are given. It is shown that shells of this type behave as two independent systems of arches at right angles. This result makes it possible to reduce the problem to one of arch analysis and develop an approximate bending theory.

Various instability modes of the fixed base column

D. A. Sawyer, *Proceedings*, ASCE, V. 86, ST7, July 1960 (Reprinted as *Technical Paper* No. 191, Florida Engineering and Industrial Experiment Station, Gainesville, 1960, 19 pp.)

Equations are presented for the critical buckling load of a column, fixed at its base and loaded at its upper end, under a variety of conditions. The case of the column loaded at its upper end through a frictionless roller is believed to be presented here for the first time. A column loaded at its upper end through a pinned link has appeared rather obscurely in the literature. This case is developed and expanded for comparison with the similar case of loading through a roller. The columns of a lift-slab operation are loaded by hanging links. Equations that have possible application in this field are derived.

General theory for the calculation of reinforced concrete with special regard to the properties of the reinforcement at fracture (in Scandinavian)

H. Granholm, *Transactions*, Chalmers University of Technology (Gothenburg), No. 209, 1959, 228 pp.

Applied Mechanics Reviews V. 13, No. 11, Nov. 1960

Starting with a description of the more recent results in testing of concrete beams and columns, the behavior of concrete in compression is discussed. Formulas for the failure moment, etc., are deduced using an idealized constant compression zone. The stress and the depth of the compression zone are found by requiring that the calculation shall give moments, etc., identical to the results when using the actual compression and strain in concrete.

The results given by this method are compared with those from a number of tests. The agreement is fairly good.

Further on the treatise contains some interesting reflections on the factor of safety. Factors include variation in concrete and steel quality, probable increase in loads in the future and type of loads.

The last part of the book gives simple formulas for dimensioning of concrete beams and other elements.

Materials

The effect of calcium chloride upon the corrosion of steel in concrete (Die Auswirkung von Kalziumchlorid auf das Korrosionsverhalten von Stahl in Beton)

A. Bazumel, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 6, June 1960, pp. 256-259.

Reviewed by Ferdinand S. Rostasy

The protection of steel against corrosion in concrete is not due to the fact that oxygen is prevented to contact the steel. It is a thin layer of iron oxide which prevents the corrosion and makes the steel passive. If this layer is destroyed corrosion starts despite a dense concrete cover. Such destruction is possible through the activating effect of calcium chloride. Author describes a method which permits, after short time, judgment as to whether steel embedded in concrete made of a certain cement will be attacked. Corrosion depends on the cement type. High-alumina content of the cement is beneficial because the dangerous chlorine ions are to a great extent chemically bound as Friedel's salt. If concrete is cured moist increasingly more chlorine ions will be bound, while during dry storage they stay active. A decrease of the water-cement ratio increases the danger of attack. Corrosion will be possible if there are differences in moisture in the structure, or if repair work is done with fresh concrete having a high $CaCl_2$ admixture, or if two types of cement are used in a two-layer job, the one having

a low C_3A content. Several types of corrosion of prestressing steel are described. Maximum admixtures of $CaCl_2$ for reinforced concrete for different types of cement are given.

Some causes of bloating in expanded clay and shale aggregates

R. D. HILL, and D. N. CROOK, *Australian Journal of Applied Science* (Melbourne), V. 11, No. 3, Sept. 1960, pp. 374-384

AUTHORS' SUMMARY

The mechanism of bloating has been studied in a number of clays and shales which could be bloated in the rotary kiln. The most common gas-producing reaction was found to be the reduction of iron oxide to the ferrous state; in the samples studied the evolution of carbon dioxide from carbonates was not important. The ferrous oxide can be combined as magnetite or fayalite, or may be dissolved in a glass. Elemental sulfur derived from the reaction of ferrous sulphide with oxygen was also a cause of bloating. Requirements for good bloating are from 3 to 10 percent of alkali and alkaline earth oxides and from 5 to 15 percent of iron oxide.

Some effects of aging on the surface area of portland cement paste

C. M. HUNT, L. A. TOMES, and R. L. BLAINE, *Journal of Research*, National Bureau of Standards, V. 64A, No. 2, 1960, pp. 163-169

AUTHORS' SUMMARY

A hardened cement paste cured at room temperature, from which part of the evaporable water has been removed by vacuum drying, has been studied. The surface area has been shown to decrease with time depending on the amount of evaporable water left in the paste. This change is the opposite of that usually observed during hydration and probably represents some colloidal growth phenomena analogous to aging observed in other colloidal gels. Both water vapor and nitrogen adsorption measurements have been used to show the effects of aging in cement paste. Wet or dry paste is shown to undergo less change than paste of intermediate evaporable water content, so that if surface area after storage is plotted as a function of evaporable water content, a curve with a minimum is obtained. With increasing storage temperature there is some indication that this minimum might shift towards lower water content. Aging is shown to occur during the initial drying of a cement paste, so that even the initial surface area of a cement paste depends on the manner in which the paste has been dried.

Modern applications of asbestos-cement (in Dutch)

D. SALOMONS, *Cement* (Amsterdam), V. 12, No. 8, Apr. 1960, pp. 712-717

Reviewed by JOHN W. T. VAN ERP

The historical background and properties of asbestos are reviewed and the use of its fibers in combination with cement. Properties of the final product are described, also manufacturing processes and the many products into which it is fabricated among them: flat sheets; corrugated, textured, and colored sheets; tubing; square ducts; and insulating panels. Its widespread use is due mainly to its durability and low cost. Originally used for industrial and utilitarian construction, it is used more and more for high quality work such as architectural applications of which many striking examples are shown.

Pneumatically applied air-entrained concrete (in Dutch)

H. J. STOKVIS, *Cement* (Amsterdam), V. 12, No. 10, Aug. 1960, pp. 894-896

Reviewed by JOHN W. T. VAN ERP

Describes properties and use of mortars and concrete called Aerocem, applied by airgun. Air-entraining agent called Teepol 480 is used, in greater quantities where good insulating property is desired. Good fire resistance, adhesion, low water absorption, resistance to frost damage are obtained. Successful repair of deteriorated structures is described. Wire mesh is used in thicknesses over 1¼ in., as is also done for all encasement of structural steel. Materials testing laboratory's data are given.

Method for producing cement of high early strength (in Hungarian)

Report No. 114, Építöanyagipari Központi Kutató Intézet (Central Research Institute for the Building Materials Industry), 26 pp.
HUNGARIAN TECHNICAL ABSTRACTS V. 12, No. 3, 1960

The report deals with the finish-grinding of various cements, testing the granulometric composition as well as the strength of the ground products. The effect of 60-micron and 3-micron cement fractions was investigated by means of substitution with glass powder. The 60-micron fraction may be considered inert from the viewpoint of both early strength and 28-day strength. The rate of solidification of the portion below 3 microns was very high, however final strength was low. Investigations on finish-grinding and on the individual fractions showed the most valuable portion of cement to be the 3- to 30-micron fractions. A certain proportion of granules not exceeding 3 microns was required as well to obtain high early strength. An adequate granulometric composition was obtained by grinding clinker in a closed cycle or by the finish-grinding of cement in a vibrating mill.

Use of small specimens in cement testing

J. A. FORRESTER and R. A. KEEN, *Journal of Applied Chemistry* (London), V. 10, Part 9, Sept. 1960, pp. 358-365
HIGHWAY RESEARCH ABSTRACTS V. 30, No. 10, Nov. 1960

It is sometimes necessary and often advantageous to make tests of cement on small specimens, and several techniques have been developed. The various methods which have been used and have been described are summarized.

There is no standard method for the testing of small specimens, and each technique must be judged on its own merits. Half-inch vibrated mortar cubes can be used for comparing hydraulic values of cements when a limited amount of material is available, or when an inexpensive testing procedure is required for purposes of control or preliminary testing. These small cubes can also produce quick results when the deterioration in strength of hydrated cements in aggressive solutions is studied.

Where larger specimens can be tolerated and a full-sized testing machine is already available, 1-in. mortar cubes are probably to be preferred for comparing hydraulic values of cements.

The use of mechanical methods of mixing and compaction by means of a jolting table are possibilities for future developments in small-scale testing.

Local pozzolana used in concrete for Koombooloomba Dam

R. V. GIPPS and A. H. BRITTON, *Civil Engineering Transactions* (Sydney), V. CE2, No. 2, Sept. 1960, pp. 65-77

Detailed description, supported by experimental results, of the use of a natural pozzolan as a partial replacement for cement in the concrete for Koombooloomba Dam, Australia.

Part I describes the testing, manufacture, and production costs of the pozzolan.

Part II outlines the practical aspects of the manufacture and use of Tully pozzolan and details results of investigation work carried out in the Koombooloomba Laboratory and by other authorities during construction of the dam.

ASTM standards on mineral aggregates and concrete (with selected highway materials)

ASTM COMMITTEE C-9 and ASTM COMMITTEE D-4, American Society for Testing Materials, Philadelphia, 9th Edition, Oct. 1960, 396 pp., $5.75

This book includes all of the standards prepared by ASTM Committee C-9 on Concrete and Concrete Aggregates, but only those for aggregates and other selected highway materials developed by ASTM Committee D-4 on Road and Paving Materials. In addition it contains pertinent specifications for cement which are under the jurisdiction of Committee C-1 on Cement.

The book contains 134 standard specifications, methods of test, recom-

mended practices, and definitions of terms for mineral aggregates, concrete, concrete curing materials, expansion joint fillers, reinforcing steel, paving block and brick, and bituminous and nonbituminous road materials. Of these standards 50 are new, revised, or have had their status changed since the 1958 edition was published.

Hydrothermal reactions between lime and aggregate fines

H. G. Midgley and S. K. Chopra, *Magazine of Concrete Research* (London), V. 12, No. 35, July 1960, pp. 73-82

Authors' Summary

Hydrothermal reactions between lime and aggregate such as pulverized fuel ash, expanded colliery shale, ground quartz, granulated and foamed blast furnace slag have been studied. The compressive strength and mineralogy have been determined for various mixtures of lime and aggregate fines autoclaved at 160 psi for 2, 6, 16, and 48 hr. The cementitious minerals identified were: tobermorite (calcium silicate hydrate) and a hydrogarnet when the aggregate was pulverized fuel ash or shale; tobermorite and zonotlite when it was quartz; and poorly crystalline tobermorite (tobermorite gel), dicalcium silicate alpha-hydrate, and a hydrogarnet when the aggregate was slag glass.

When dicalcium silicate alpha-hydrate was the predominant phase, the strength of the blocks was lower than when tobermorite was the main phase.

Pavements

Epoxy resin for structural repair of concrete pavement

Wilson L. Davis and Eugene Pinkstaff, *Proceedings*, ASCE, V. 86, CO1, Feb. 1960, pp. 55-70

Authors' Summary

Describes tests performed in the North Central Division Laboratory of Corps of Engineers in formulating epoxy resins that will satisfactorily repair surface defects in portland cement concrete pavements, that will bond the hardened surfaces of concrete together, and that will bond new concrete to existing concrete. It describes also the field application of the resulting epoxy resins.

Automatic surveying system measures runway roughness

R. S. Brown, *Electronics*, V. 33, No. 25, June 17, 1960, pp. 54-56

Reviewed by Aron L. Mirsky

Description of automatic surveying device developed for the Air Force, which is at least ten times faster than any system heretofore known. Using a light beam projector and profile measuring device, system measures and records profile variations at intervals of 6 in., with dynamic accuracy better than ¼ in. at peak speeds of 15 measurements per sec.

British Standard Code of Practice CP 2006 (1960). Traffic bearing structures — pavings

Council for Codes of Practice, British Standards Institution, London, 1960, 204 pp., 30s

Road Abstracts
V. 27, No. 8, Aug. 1960

This code deals with the design, construction, and maintenance of pavings for use by all types of road vehicles, pedestrians, and aircraft. It is presented under the following headings: Part 1. Design and construction of foundations: introduction; main principles; factors affecting subgrade preparation; subgrade preparation; methods of estimating pavement thickness; construction of bases and sub-bases; concrete haunches; pavings on peat. Part 2. Concrete roads and airfields: introduction; materials; concrete; design of concrete pavement slabs; methods of construction; maintenance and repair of concrete pavements. Part 3. Surfacings other than concrete: general; bituminous surfacings; grouted macadam; water-bound surfacings; surface dressing using tar, bitumen, tar-bitumen mixtures or bitumen emulsions; machine laying of surfacing materials other than concrete. Part 4. Footways, cycle tracks and verges: factors affecting design; types of construction; preparation of subgrade; base; surfacings;

verges; hard shoulders; curbing; footway crossings. Part 5. Surface water drainage: roads; airfields. Appendices are included on concrete mix design and on standards of riding quality of road and runway surfaces and their assessment.

Successful use of prestressed concrete slabs for road construction in communist countries (in German)

SZACILLO, *Brücke und Strasse*, V. 11, No. 10, 1959, pp. 374-376

ROAD ABSTRACTS
V. 27, No. 6, June 1960

Details are given of the use of prefabricated slabs in the USSR, China, Hungary, Czechoslovakia, and Poland. (This is a translated version of an article which appeared in the Polish journal *Drogownictwo*, Warsaw.)

Triple-batch paver teamed with slip-form

H. K. GLIDDEN, *Roads and Streets*, V. 103, No. 11, Nov. 1960, pp. 45-46, 72,76

Brief report on the slip-form paving method used on 15 miles of Interstate 25, near Denver, Colo. The pavement consists of two to 24-ft lanes of concrete, 8 in. thick and nonreinforced. Three factors are necessary for efficient slip form usage: (1) uniform trackline grade, (2) uniform consistency of concrete, and (3) operation must be continuous.

Warping stresses and deflections in concrete pavements: Part II

JOSEPH F. WISEMAN, MILTON E. HARR, and GERALD A. LEONARDS, *Proceedings*, Highway Research Board, V. 39, 1960, pp. 157-172

AUTHORS' SUMMARY

A solution has been obtained for stresses and deflections of concrete slabs on ground subjected to temperatures and/or moisture contents which decrease linearly with the depth of the slab.

The derived equations were programed with the aid of a Compiler and solutions were obtained from the Datatron 204 digital computer.

Nomographs are presented which give the ratios of the maximum stress to the modulus of rupture for a wide range of environmental gradients and for various combinations of slab sizes and thicknesses. The solutions are obtained for the critical condition which develops when the slab is only partially supported.

A 12 x 25-ft slab 8 in. thick, resting on a soil-cement subbase, was appropriately instrumented and subjected to temperature gradients under controlled conditions. Comparisons between computed and measured deflections confirm the validity of the new theory.

McPherson test road: Tenth-year report

W. M. STINGLEY, JOHN D. MCNEAL, and R. L. PEYTON, *Proceedings*, Highway Research Board, V. 39, 1960, pp. 191-204

FROM AUTHORS' SUMMARY

Due to a lack of suitable stone, Kansas is compelled to use sand-gravel aggregates for concrete, and much of the concrete produced from these aggregates is subject to map cracking. In most cases this cracking can be minimized or delayed by the addition of 30 percent limestone, but this is a costly material in the western two-thirds of the state. The McPherson Test Road was built in 1949 to determine if less expensive methods might prove to be as effective.

Significant differences exist in the behaviors of the test sections. Air entrainment hastened map cracking during the first 5 years, but map cracking in air-entrained sections tended to increase at a slower rate, or cease, during the next 5 years. Cement brand, and type, is an important variable with relation to extent of map cracking, and to the time elapsed before it appeared. Limestone is still an effective deterrent to map cracking, at least with most cements used in the test. However, it appears that some cements may achieve the same degree of map crack inhibition without the limestone.

Flexural strength of test beams showed marked seasonal variations in the sections containing pozzolans, but in general increased through the 10-

year period. Test beams representing basic aggregate sections declined in strength with only minor seasonal variations, except that the Type II cement section maintained its early strength. The sections which included crushed limestone did not fluctuate markedly in flexural strength and showed small gains over the 10 years. Transverse and longitudinal cracks appeared earlier and were more extensive in some pozzolan sections, although the basic aggregate sections with considerable early cracking became so map cracked that structural cracks tended to be obscured.

Although this report covers the projected 10-year period, and is considered a summary of the final report, it is planned to keep the test under observation for the life of the road.

Final report on durability project, Michigan test road

C. C. Rhodes and E. A. Finney, *Proceedings*, Highway Research Board, V. 39, 1960, pp. 217-309

Authors' Summary

The durability project of the Michigan Test Road (built in 1940) was designed to study the effect of various factors on the durability of concrete in service. The study included both materials and operations, principally the following factors: (1) proportioning and grading of aggregates; (2) various types of additives, including plasticizers and air-entraining agents; (3) blends of portland with natural cement produced with and without a grinding aid; (4) limestone aggregates in various combinations and gradings; and (5) finishing and curing. Supplementary laboratory studies preceded and accompanied the construction and evaluation of the pavement. Several incidental studies were also carried out in connection with the construction of the project, and accelerated scaling tests were performed on the test areas during the first two winters after construction.

The most outstanding result was the early verification of the beneficial effect of air entrainment on the durability of concrete, which led to the decision in 1943 to use air-entrained concrete in all Michigan pavements. Blending plain natural cement with portland cement improved scale resistance considerably, but the effect of the natural cement was magnified when beef tallow had been added as a grinding aid. The accelerated scaling tests indicated that for the mixtures used limestone aggregates were conducive to scaling and that adding limestone dust tended to aggravate the condition rather than relieve it. In fact, the addition of fines in general produced no improvement in durability. Curing methods had little influence on ultimate durability, but the bituminous and transparent membranes caused undesirable temperature effects in the concrete. In the finishing study, brooming was moderately beneficial but not greatly superior to burlap finishing in its effect on resistance to scaling.

The relative performance of the experimental sections of the pavement during the first 17 years of service generally followed the pattern set in the early accelerated durability tests; the air-entrained concretes exceeded all others in durability and the sections with limestone aggregates were the first to require resurfacing.

Precast Concrete

Production and application of concrete, reinforced concrete and asbestos-cement pipes (Herstellung und Verwendung von Beton-, Stahlbeton- und Asbestzement-Rohren)

W. Ritter Von Meng, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 6, June 1960, pp. 245-255.

Reviewed by Ferdinand S. Rostasy

In the introduction the author gives a short historical review of the use of concrete pipes. This is followed by a description of the customary production methods, classified according to the mode of compaction such as hand rodding, machine compaction, compaction by external and spud vi-

brators, compaction by centrifugal force, and vacuum concrete method. Main requirement of the concrete is resistance against aggressive water. Concrete technological principles to guarantee this are outlined. Curing conditions are of great importance.

General construction principles for concrete pipelines are discussed, such as transport, choice of cross section, slope of pipeline, connections and branch pieces. Summarizing, the German specifications are repeated. Concrete pipes are permitted without any further protective surface for any kind of sewage having a ph-value > 6. Absolute water-tightness and absence of cracks are required. Test procedures concerning water-tightness are described.

Asbestos-cement pipes have been in use since 1899. The qualities of the asbestos, of the cements used, and of the finished product, as well as the production are described in detail. The behavior under internal pressure, external loads and in aggressive water is discussed.

Cathodic protection of prestressed concrete pipe

M. Unz, *Corrosion*, V. 16, June 1960, pp. 123-131
Battelle Technical Review V. 9, No. 9, Sept. 1960

Prestressed concrete pipe has the reinforcement either partly embedded in the concrete core or totally placed over the core under a cement mortar cover. The lines have the advantage of being suitable for cathodic protection. The feeding points must be spaced closer due to stronger attenuation.

Silos built of prefabricated elements (Prefabrication de silos a argile cuite)

W. Zalewski, *La Technique Modern-Construction* (Paris), V. 15, No. 8, Aug. 1960, pp. 314-317
Reviewed by Alexander M. Turitzin

An account is given of the construction of a series of silos of reinforced concrete prefabricated elements for the storage of baked clay in Poland. After a comparative study between the cast-in-place and the prefabricated method of construction, it was found that the latter resulted in substantial saving in labor and material. Construction with prefabricated elements permitted mechanizing the project, placing concrete in better conditions, and assembling and erecting the structure in cold weather at temperatures well below 0 C.

The author emphasizes that prefabrication does not simply consist of cutting the structure into a certain number of elements and joining them together. It must be preceded by a thorough study of the project with regard to stability and the sequence of erection.

Precast bridge beams: Standard sections

Engineering (London), V. 190, No. 4927, Sept. 23, 1960, p. 412

Standardised bridge beams in prestressed concrete

The Engineer (London), V. 210, No. 5461, Sept. 23, 1960, p. 507
Reviewed by Aron L. Mirsky

Brief descriptions, with drawings, of group of standardized precast prestressed bridge beams, designed for British highway loadings. Only two sets of forms are required, one for spans of 25-35 ft (two sections) and the other for spans up to 55 ft (five sections); these ranges differ only in depth of web, the various sections within each range differing in depth of top flange.

Prestressed Concrete

Test on a 120-ft span prestressed concrete beam

G. D. Base and R. E. Rowe, *Proceedings*, ASCE, V. 86, ST9, Sept. 1960, pp. 1-26
Authors' Summary

Tests to destruction have been carried out on a 120-ft span prestressed concrete beam similar to the beams to be used in the construction of a four-span overpass at the Chiswick rotary

junction. The tests showed that the design assumptions with regard to the loss of stress due to friction and short-term creep, shrinkage, and relaxation were valid. The behavior of the beam in the working-load range agreed excellently with the behavior predicted.

At ultimate load the beam failed by crushing of the concrete in the top flange at a deflection of 20 in. The calculated ultimate load was 6 percent less than the actual failing load; the assumptions, on which the ultimate load calculations were based, were thus fully vindicated.

Prestressed girder bridges double as heavy utility carriers

Roads and Streets, V. 103, No. 11, Nov. 1960, pp. 69-71

Two post-tensioned concrete girder bridges are constructed in New York City utilizing an interesting design feature. The spaces between the girders enclose duct areas for an array of utility lines. The girders are spaced on 11-ft 6-in. centers and are sheathed on the underside by a 6 in. reinforced concrete slab.

Use of prestressed concrete for the strengthening of the belltowers of the Chaise-Dieu Abbey, France (L'utilisation du béton précontraint dans la consolidation des clochers de la Chaise-Dieu)

A. J. Donzet, H. Mottin, and M. Hemard, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), Supplement No. 143, Nov. 1959, pp. 1-25

Reviewed by Henri Perrin

In 1949 signs of collapse were observed in the two pillars of the first bay of the Abbey, founded in 1043. The nave is rather low and the aisles are as high as the nave. A big portion of the loads were carried by inside pillars of polygonal section. After an emergency shoring, it was decided to divert some of the excess loads to the buttresses which shoulder the belltowers.

Two trusses of prestressed concrete were built above the ceiling; their spans are not equal since the transverse walls are not parallel.

Numerous measurements showed that the transfer of load was performed according to the design.

Some aspects of durability and volume change of concrete for prestressing

Paul Klieger, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 3, Sept. 1960, pp. 2-12

Author's Summary

Describes laboratory tests performed to evaluate the resistance to freezing and thawing and to de-icer scaling of concretes typical of those used in the fabrication of prestressed elements. Both non-air-entrained and air-entrained concretes made with Type I and Type III portland cements were included in this study. The influence of various curing procedures was evaluated, both at normal temperature and elevated temperature. In addition, the effect of a prestress producing 350 to 400 psi compression in the concrete on the resistance to freezing and thawing was determined.

Lateral buckling strength of prestressed concrete beams (Stabilité élastique des poutres en béton précontraint a l'égard du déversement latéral

P. Lebelle, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 141, Sept. 1959, pp. 779-831

Reviewed by Henri Perrin

The transaction deals, in connection with prestressed concrete, with: elastic stability of an asymmetrical I-beam with constant cross section and elastic torsional restraint; lateral bending of beams; stability of suspended beams; bracing of beams with horizontal guys; and effects of restraint on lateral bending and stability of continuous beams.

The relationship between the dead load of the beam and the critical tilting load does not yield a reliable safety factor during erection since buckling may occur without any increase of the applied load. This leads to an investigation of the various factors, such

as inherent crookedness, unknown eccentricity, or torsional restraint, which may cause failure. By assuming reasonable values for each of them one is able to determine the stresses likely to be reached during erection. A few graphs complete a thorough mathematical investigation.

Behavior of prestressed concrete under dynamic loading

B. W. MARSHALL and A. M. OZELL, *Journal*, Prestressed Concrete Institute, V. 5, No. 4, Dec. 1960, pp. 74-84

Describes the dynamic testing of a prestressed concrete diving board 20 in. wide, 2⅝ in. thick, tapered to 1½ in., and 16 ft long. The free end was cantilevered 11 ft. The board was pretensioned with six 7/16 in. strands spaced 3 in. on centers. Data are presented on properties and responses of the board to the action of diver and static tests.

Prestressed beams shore drydock walls

Engineering News-Record, V. 165, No. 16, Oct. 20, 1960, pp. 42-43

Brief description of the drydock for the Webster St. Tube which will carry automotive traffic under the estuary between Oakland and Alameda, Calif. The walls of the 473-ft drydock are of timber lagging supported by 50 ft prestressed concrete soldier beams. Sheetpiling keeps estuary water from drydock excavation.

Ultimate strength of pretensioned I-beams in combined bending and shear

R. SETHUNARAYANAN, *Magazine of Concrete Research* (London), V. 12, No. 35, July 1960, pp. 83-90

AUTHOR'S SUMMARY

Summarizes conclusions drawn from tests on pretensioned I-beams without web reinforcement. The factors that govern the ultimate strength of such beams in combined bending and shear, and their influence on the mode of failure, cracking, and the ultimate load are studied. A simple theoretical analysis is presented for evaluating the ultimate strength of I-beams failing by crushing of the web. The theoretical expression is compared with the test results of the author and of other workers and shown to result in a fairly safe and reasonably accurate estimate of the ultimate load. An empirical equation is suggested to determine the inclined cracking load and the collapse load for diagonal tension failures.

Prestressed piles stand unusually hard driving

Contractors and Engineers, V. 57, No. 9, Sept. 1960, pp. 70-73

Precast beams, joists cast into building frame

Contractors and Engineers, V. 57, No. 9, Sept. 1960, pp. 74-76, 81

Plant is semiportable

Contractors and Engineers, V. 57, No. 9, Sept. 1960, p. 82

Prestressed concrete piles required 25,000 blows to be driven 180 ft to coral and sand bearing for Ala Moana office building, Honolulu. The 16½- and 18-in. octagonal prestressed piles were designed for loads of 75 and 125 tons spaced and 24- and 30-ft centers.

A special boom, mounted on the materials tower, and a pair of dollies were used to hoist some 1325 precast, prestressed beams and joists weighing 6500 lb each.

A unique feature of the project was that the four pretensioning beds, 12 ft wide and up to 460 ft long, were designed to be moved when the job was completed.

Cut slope is prestressed with beams and anchors

Engineering News-Record, V. 165, No. 16, Oct. 20, 1960, pp. 52-53

Emphasizes construction difficulties in stabilizing a cut slope at end of bridge in Walnut Creek, Contre Costa County, Calif. Bridge is 291 ft long, five-span concrete structure supported by abutments and four reinforced concrete, two-column bents. Three reinforced concrete beams 1 ft 3 in. thick and 5 ft wide are prestressed with 1⅛

in. diameter steel rods, 75 ft long. Fourteen pairs of steel rods, stressed to 50 tons, load each beam giving the soil a resulting pressure up to 3 tons per sq ft.

Concrete for prestressed concrete structures (Der Beton im Spannbetonbau)

R. Joosting, *Schweizer Archiv* (Zurich), V. 25, No. 9, Sept. 1959, pp. 347-351
Reviewed by Henri Perrin

The requirements for high-quality concrete as used in a prestressed concrete structure are reviewed: the main characteristics of a few admixtures are critically exposed, while the conditions of compatibility between cement and admixture are outlined.

Long span prestressed concrete folded plate roofs

John C. Brough, Jr. and B. H. Stephens, Jr., *Proceedings*, ASCE, V. 86, ST10, Oct. 1960, pp. 87-108
Authors' Summary

The design and construction of long span prestressed concrete folded plate structures are presented. The economics of such design is indicated by including actual construction costs of four roof structures. A simplified design procedure is outlined and a typical case presented. Field experience with construction methods utilized and recommendations for field inspections are included based on the authors' observations.

Safety against development of cracks in edge zones of periodically prestressed deep beams (Die Rissesicherheit in den Randzonen periodisch vorgespannter Scheiben)

W. Schleeh, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 4, Apr. 1960, pp. 93-95
Reviewed by Rudolph Szilard

The prestressing of deep beams using numerous individual prestressing cables produces tensile stresses in the direction perpendicular to the prestressing. To prevent the development of cracks, prestressing in two directions is required. The paper deals with the determination of such stresses utilizing the Fourier-series solution of the differential equations. A numerical example shows the practical use of the derived formula.

Properties of Concrete

Influence of the type of cement on its cracking tendency

A. De Sousa Coutinho, *RILEM Bulletin* (Paris), No. 5, New Series, Dec. 1959, pp. 26-40
Author's Summary

This paper is a sequence to one published 4 years ago on the cracking tendency of cements, mortars, and concretes. It is mainly concerned with the cracking tendency of natural cement, standard portland cement, high-early-strength cement, and aluminous cement.

The cracking tendency is measured by the cracking of mortar rings cast around metallic rings and by the stress set up in these restrained shrinkage rings.

From the tests described the following conclusions are drawn: (1) Natural cement, as well as concrete prepared with it, has an extremely low cracking tendency. (2) High-early-strength portland cement has a high cracking tendency due to its slight creep (or relaxation). Concrete prepared with it has a greater cracking tendency than that prepared with normal portland cement. (3) Aluminous cement has the greatest cracking tendency of all the cements, due to its extremely small creep and to the very large increase of its initial shrinkage. (4) Normal portland cement has a cracking tendency intermediate between that of natural cement and of aluminous or high-early-strength cement. But, its cracking tendency grows higher as its strength increases.

A relationship is deduced between the strength of the cement and its creep. In the previous paper, a relation was established between the creep of concrete and the rupture stress of this concrete.

Based on this study it is stated that in a concrete, the greater the rupture

strength, the lower the creep and the greater the stress set up in restrained shrinkage. But, when a given parameter changes—such as the cement or the water dosage, the maximum size of aggregate, etc.—the rupture stress increases more rapidly then the stresses set up in a specimen under restrained shrinkage, and the possibilities of cracking are reduced accordingly.

Hence, the general conclusion: in concrete, the greater the strength the lower the cracking tendency; in cement, the greater the strength, the higher the cracking tendency.

On the specific surface of pulverulent matters (Sur la Surface Specifique des Matieres Pulverulentes)

A. Ilantzis, *Revue des Matériaux* (Paris), No. 534, Mar. 1960, pp. 59-66
Reviewed by Phillip L. Melville

The theoretical basis of the Blaine air permeability apparatus (ASTM C 204) is reviewed and a maximum error of 2.7 percent resulting from an absolute determination of its constant is computed. The computation is checked using different liquids and geometrical forms. The constant of an apparatus is computed as accurately as if a standard powder had been used.

Plasticity and plastic flow (Plasticité et fluage)

G. Colonnetti, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 142, Oct. 1959, pp. 1075-1087
Reviewed by Henri Perrin

Author, after restating the classical distinction between elastic and plastic deformations, emphasizes how far from reality this distinction is since both types of deformation react on each other. Furthermore, the deformations are also functions of conditions which may have been previously imposed on the material. This stems from the concept of a "heredity factor: f" (Volterra's factor), assumed to be constant for a homogeneous body.

The notion of hysteretic cycles is also presented for a homogeneous body subject to an alternate loading condition. Author applies then his proposed theory to a homogeneous prismatic beam subject to simple bending, also to statically indeterminate beams and finally to heterogeneous bodies.

New machine for the fatigue testing of reinforced concrete beams, and some preliminary results

W. J. Larnach, *Magazine of Concrete Research* (London), V. 12, No. 36, Nov. 1960, pp. 171-176
Author's Summary

Describes the construction and operation of an hydraulically operated fatigue machine of 5 tons capacity for tests in bending or compression. The machine proved capable of long periods of service with the minimum of attention.

A series of fatigue tests in bending on normally reinforced concrete beams indicated that high proportions of the static failing load could be applied as many as 300,000 times without significantly altering the ultimate strength of the beams.

A test method for air entrainment of standard Ottawa sand

Martin R. DeFore and Harold J. Corah, *ASTM Bulletin*, No. 248, Sept. 1960, pp. 48, 53-55
Authors' Summary

Tests indicated that standard Ottawa sand from different lots contributed in varying degree to the amount of air entrained in the air entrainment test for portland cement. Washing and drying the sand resulted in lower values for air entrainment in cement-sand mortars, especially with non-air-entraining cements. The weights of 400-ml samples of sand-water mixtures were used to evaluate the air-entraining properties of the sand. Studies of different methods of washing and drying the sand revealed a procedure whereby better reproducibility was obtained in the determination of the air content of the mortar. Repeated wetting and drying may be necessary. On the basis of these tests it appears that 400 ml of a mixture of washed standard sand with 25 percent water should weigh at least 792 g when mixed and placed by standard test procedures.

Workability of concrete

L. J. MURDOCK, *Magazine of Concrete Research* (London), V. 12, No. 36, Nov. 1960, pp. 135-144
AUTHOR'S SUMMARY

An analysis is made of a large number of tests for workability as measured by the compacting factor, to determine how workability is related to the grading and shape of aggregate, the mix proportions, and the water-cement ratio.

An index is developed which takes account of the influence of particle size on workability, and largely overcomes the limitations of the fineness modulus and surface area coefficients. Similarly, an angularity index is introduced which is based on a test on the aggregate and is shown to take account of its shape.

The relation between workability and mix proportions is assessed. It is established that a proportion of the water, equivalent to a water-cement ratio of about 0.25, is required to wet the cement, and that any additional water may be regarded as available to make the mix workable.

An approximate relation is given for the calculation of compacting factors, and a worked example illustrates the use of this relation for mix proportioning.

Structural Research

Buckling of arches (in German)

A. SCHWERTNER, *Acta Technica*, Academiae Scientiarum Hungaricae (Budapest), V. 26, No. 1/2, 1959, pp. 175-189
APPLIED MECHANICS REVIEWS
V. 13, No. 9, Sept. 1960

Author deduced a new method for static and dynamic calculation of beams and of buckling. The essence of the method is that a beam only loaded at its ends is replaced by an elastic point, and such points are joined in different ways parallel to each other and in series. The present paper dealing with an arch built-in at both ends, completes his preceding papers on the method in question. In his examples author determined the critical load of a test-arch made from spring steel and of a reinforced concrete arch. It is unfortunate that the results obtained were not confirmed by tests.

Shear transfer can be simple

HOMER M. HADLEY, *Engineering News-Record*, V. 164, No. 21, May 26, 1960, pp. 45-46

This short article gives the results of exploratory tests to demonstrate that shear transfer to provide composite action between steel bridges and concrete decks can be simple. Two tests were run with beads or buttons of weld metal to provide shear transfer. Both tests provided unit shear stresses in excess of 400 psi and neither failed in shear. Although very limited this test indicates a composite action of an effective nature was being obtained.

Lateral buckling problems in hoisting and erection of slender beams (in Swedish)

OVE PETTERSSON, *Nordisk Betong* (Stockholm), V. 4, No. 3, 1960, pp. 231-270
Reviewed by MARGARET CORBIN

Deals with lateral buckling problems met with in hoisting and erection of slender beams and presents the basic equation for the angle of rotation ϕ in the position of equilibrium involving a spatial deflection which is associated with lateral buckling. This equation which is valid on the assumption that the material is elastic and homogeneous, is solved to determine the lateral buckling load on a beam which is suspended by means of ropes, and which is acted on by dead load, in the following cases: (a) beam with cross section which is constant and free to warp; (b) beam with cross section which is free to warp and varies linearly in depth; (c) beam with cross section which is constant, monosymmetrical, and not free to warp. The lateral buckling loads of beams on supports prevented from torsion are also determined in a somewhat more general case of loading where the dead load is combined with a uniformly distributed load applied at any arbitrary height above the axis of gravity of the beam.

Reinforced concrete shear wall assemblies

JACK R. BENJAMIN and HARRY A. WILLIAMS, *Proceedings*, ASCE, V. 86, ST8, Aug. 1960, pp. 1-32

AUTHORS' SUMMARY

Gives the results of an investigation of one-story and two-story reinforced concrete shear wall assemblies. Tests were made on four one-story models with parallel shear walls connected by a reinforced concrete diaphragm. Loads were applied to the diaphragm, and three separate diaphragm tests were then made. Following these tests, two one-story specimens with face walls were tested. Finally, two two-story models were tested.

Theoretical studies were made for all tests and practical design approximations are suggested.

Studies of endurance limit and impact strength of light ferro-concrete applied in the design of bridges and other structures (in Russian)

M. A. YAKUBOVICH, *Sbornik Trudov Tbilissk* (Moscow), No. 30, 1956, pp. 96-122
APPLIED MECHANICS REVIEWS Oct. 1959

Tests carried out by the author, in cooperation with M. V. Kandelak, on repeated loading (up to 2,000,000 cycles) showed that the endurance of pumice concrete to a compressive stress of 60-80 kg per sq cm is no lower than the endurance of heavy concrete of a higher grade. In 1952 Sukhum carried out endurance tests on a spongolite ferro-concrete beam with a span of 30 m made from Russian grade M-170 concrete. With an amplitude of vibration which corresponded to the deflection of the structure due to the working load, the beam (at a frequency of 3-4 Hertz) withstood 2×10^6 test cycles without suffering injury. When the amplitude of vibration was doubled, fracture took place after 280,000 cycles (jointly with the rupture of the tensile reinforcement in the center of the span). It was established that for automobile road bridges 4-17 m wide made with pumice, tufa, and spongolite of grades M-150 to 170, the dynamic coefficient ensuing from the action of the working load was found to have a mean of 1.2. Test results from structures subjected to impact loading are presented; also presented are values of the relative deflections, due to working load, of beam and arch type light ferro-concrete bridges. It would seem that the "generalized" formulas recommended by the author cannot pretend to claim great accuracy or universality of application, because the formulas do not take into account such factors as creep and shrinkage of lightweight concrete, or the relaxation of stresses in the reinforcement.

Report of flexural test on 8 ft length of 36-in. reinforced concrete pipe—ASTM C 76-56, Table 1

C. J. CLAYWORTH, *Concrete Pipe News*, V. 12, No. 7, July 1960, pp. 75-79

Purpose of test was to provide empirical criteria for evaluating the ability of 8 ft long, 36 in. reinforced concrete pipe to resist bending moment. Ultimate failure of specimen was result of progressive crushing at the ends, over the supports.

Crack and deformation similitude in reinforced concrete

J. FERRY BORGES and J. ARGA E LIMA, *RILEM Bulletin* (Paris), No. 7, New Series, June 1960, pp. 79-90

AUTHOR'S SUMMARY

Similitude conditions for crack and deformation in reinforced concrete are both theoretically and experimentally, analyzed, it being sought to define, the conditions to be fulfilled by the models, in order that sufficiently reliable representation of the phenomena to be studied can be obtained.

It is shown that it is possible, as a rule, to reproduce usual structures by makings models to adequate scales with the same materials as the prototypes, i.e., concrete and steel. Phenomena of cracking deformation and rupture can be studied in the models obtained. This is confirmed by the results presented which concern bending

tests of similar beams to different scales.

Test on reinforced concrete models are useful, not only for solving definite design problems, but also for obtaining data for developing theories, notably on cracking phenomena.

Lateral buckling of slender reinforced concrete beams

A. Sievv, *Magazine of Concrete Research* (London), V. 12, No. 36, Nov. 1960, pp. 155-164

Author's Summary

A brief summary is given of the theory of buckling for homogeneous materials, with special reference to reinforced concrete, account being taken of variations in shape and in loading. The method involves the determination of the flexural and torsional rigidities in the uncracked, cracked elastic, and cracked plastic states.

Tests were made on two series of beams: six of rectangular section (1½ x 16 in.), and four inverted L-beams.

It is concluded that the span-width ratio is not the only factor affecting buckling; the width-height ratio and the percentage of reinforcement and its distribution are of equal importance.

Cracking of reinforced concrete structures (in Hungarian)

G. Nemeskeri-Kiss, *Mélyépítéstudományi Közlöny*, V. 9, No. 11, 1959, pp. 514-530

Hungarian Technical Abstracts V. 12, No. 3, 1960

Earlier reinforced concrete design specifications for reinforced concrete railway bridge structures aimed at the complete elimination of cracking. This resulted in considerable structural depth, the load-bearing capacity of the building materials was not fully exploited, and reinforced concrete structures became oversized. On the basis of theoretical considerations relating to the development of cracks, a series of tests was carried out to study the behavior of deformed, helically ribbed Hungarian reinforcing steels. In these experiments the development of cracks in simply supported reinforced concrete beams subject to static as well as dynamic bending loads was examined. Cracks were recorded immediately after each loading cycle; width of the cracks, deflection of the beams, and magnitude of the strains were measured. Experiments verified that under identical conditions the width of cracks developed in concrete beams reinforced with deformed steel is significantly smaller than in case of smooth round bars. Experience has proved that the maximum permissible span of railway trough bridges can be increased and the structural height of bridge structures of different spans can be reduced.

Use of models of reinforced concrete structures

Emanuele Fumagalli, *Magazine of Concrete Research* (London), V. 12, No. 35, July 1960, pp. 63-72

Discusses the use of models to determine the behavior of full-scale structures in reinforced concrete, with particular reference to recent work done at the ISMES laboratories in Bergamo, Italy.

Distribution of stress in anchor blocks

R. E. Rowe and J. Zielinski, *Engineering* (London), V. 189, No. 4899, Mar. 11, 1960, pp. 347-348

Reviewed by Aron L. Mirsky

Experiments on simple prisms of square section subjected to various single concentrated loads, simulating the anchorage zone of post-tensioned concrete members, show that the maximum transverse tensile stresses are considerably larger than, and disposed differently from, the theoretical values. Type of anchorage (embedded or external) and method of anchoring the wires, does not significantly affect the stresses or the ultimate load; the dominant factor is the ratio of loaded area to cross-sectional area of the prism. The percentage of reinforcement is significant when the contact stresses are smaller than about 1.9 times the cube strength of the concrete.

A full report, including a design method based on this investigation, is to be published by the Cement and Concrete Association, London.*

* Rowe, R. E., and Zielinski, J., "An Investigation of the Stress Distribution in the Anchorage Zone of Post-Tensioned Concrete Members."

Strength and stability of concrete constructions (in Russian)

V. N. Gornov, Gosstroiizdat, Moscow, 1957, 120 pp. 5r 45k

Applied Mechanics Reviews V. 13, No. 11, Nov. 1960

Results are published of the author's work on investigations of the strength of deflecting and eccentrically compressed lightly reinforced and pure concrete construction components. Studies were made of a light concrete with a volumetric weight of 1.6 to 1.75 tons per cu m mixed with "Kashirsk" slag and of a heavy concrete with a volumetric weight of 2.1 to 2.5 tons per cu m mixed with granite chips. The mean relative extensibility of the concrete for momentary deflection tests on 75 beams was 1.8×10^{-4}; the limiting compression of the slag concrete for momentary axial compression on 24 prisms was 15×10^{-4}. Because of the creep in the concrete the limit of deflection on 28 concrete beams, under loads for a duration of 114 24-hr periods continuously, proved to be approximately twice as large for the heavy concrete and three times as large for the slag concrete as the deflections for momentary loads. Observations over 4 months disclosed a relative shrinkage growth in slag concrete reaching a magnitude of 4×10^{-4}.

Formulas are given for the determination of the carrying capacity of compressed concrete components, the formulas being based on the assumption that the limiting state is characterized by the appearance of the first power cracks in the danger section of the component, which takes into account the initial eccentricity in the application of the load and the limiting extensibility of the concrete. A formula is also furnished for the determination of the curvature of the deflecting component of the concrete when consideration is being given to the effect of prolonged loading. A comparison of the formulas derived by the author with the formulas embodied in the existing building standards and rules of SN and P shows that the SN and P formulas give lower values for the breakdown load while the author's formulas agree well with the experimental data. The creep in concrete reduces the carrying capacity of columns, which justifies to some extent the provisions of the SN and P formulas.

General

Possible applications of nuclear magnetic resonance in cement and concrete research

Paul Seligmann, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 3, Sept. 1960, pp. 20-31

Author's Summary

Many techniques arising from atomic and nuclear physics are finding applications in the solution macroscopic physical and chemical problems. The principles of the recently developed nuclear magnetic resonance techniques are described. Their potential application in cement and concrete research, particularly in studies of the classes of water present in the hydrated cement paste are examined. The results of the only published work on applications to cement paste are outlined.

Reinforcing wire used as heating system

Concrete, V. 68, No. 12, Dec. 1960, pp. 14-15, 25

Revolutionary heating system, in which the reinforcing wire fabric in a concrete floor slab also serves as the electric heating element. This heating system, shortly to be tested near Minot, N. D., will be subjected to 40 degree temperatures. Results are still inconclusive but prospects are promising.

Wind pressures in various areas of the United States

GUTTORM N. BREKKE, *Report* 152, Building Materials and Structures, National Bureau of Standards, U. S. Department of Commerce, Government Printing Office, Washington, D. C., Apr. 1959, 8 pp., 15 cents
BRI ABSTRACTS
June 1959

A procedure is described for developing a wind-pressure map that shows minimum resultant wind pressures (30 ft above ground) for design purposes. It is based on records of annual fastest-single-mile wind speeds at 155 U. S. Weather Bureau stations. In computing resultant wind pressures from these data, allowance was made for gusts and for building shape. A table of design wind pressures at various heights from less than 30 to more than 1200 ft above ground is provided for use with the map. Principal sources of strong winds affecting building constructions are reviewed briefly.

Construction in Russia: Report covering an educational trip (Bauen in Russland, Bericht über eine Studienreise)

F. LEONHART, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 4, Apr. 1960, pp. 81-88
Reviewed by RUDOLPH SZILARD

An interesting summary of the newest developments in reinforced concrete technology in the Soviet Union. The new 7-year plan requires the erection of 15 million new apartments. For such an enormous building program, standardization, precasting, and prestressing of all elements is mandatory. The same basic principles have been used in construction of factories, warehouses, and other industrial buildings. The use of structural steel has been reduced to a minimum. Prefabrication and prestressing even for long span monumental type bridges (like the famous Krasnoholmski Bridge in Moscow) is the prevailing trend for new bridge constructions. Reinforced concrete has been used in building a 1660 ft television and radio tower, which has been prestressed longitudinally to take the wind load. Structural research is on very advanced level covering materials, response to dynamic loading, prestressing without bond, the effect of high temperatures, and new design.

Drawing by sandblasting

Progressive Architecture, V. 41, No. 12, Dec. 1960, pp. 154-157

Uniformly distributed aggregate concrete was sandblasted at Oslo, exposing a homogeneous surface pattern. New technique, called *Naturbetong*, has satisfactory compressive strength. Additional advantages are: all lift marks and construction joints can be erased; possibility of voids and other irregularities is reduced, and patching does not leave visible marks.

1958 Progress Report of the Laboratories du Batiment et des Travaux Publics (Compte rendu des recherches des Laboratoires du Batiment et des Travaux Publics effectués en 1958)

R. L'HERMITE, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 139-140, July-Aug. 1959, pp. 737-777
Reviewed by HENRI PERRIN

The research projects of the Laboratoires du Bâtiment et des Travaux Publics carried on during 1958 are summarized in the present publication. Some of them have not yet reached their final stage; their description is however useful since it gives a good account of the performed progress and indicates the trends of future research. The main items of interest are: cements, concrete, masonry, soil mechanics, basic research.

Note

Copies of books and articles reviewed are not available from ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In some cases ACI can furnish addresses of publications added later.

ACI NEWS LETTER

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memo from the President:

ACI in Action

THE OBJECTIVES OF ACI in stimulating research and in finding the best ways to do concrete work are generally understood and perhaps a little taken for granted by many of us.

Most ACI members are aware of the working technical committee organization of ACI which embraces over 50 committees and more than 600 persons working on various assignments from the Technical Activities Committee which provides over-all guidance. Ten of these committees described their progress at one session during the recent convention in St. Louis.

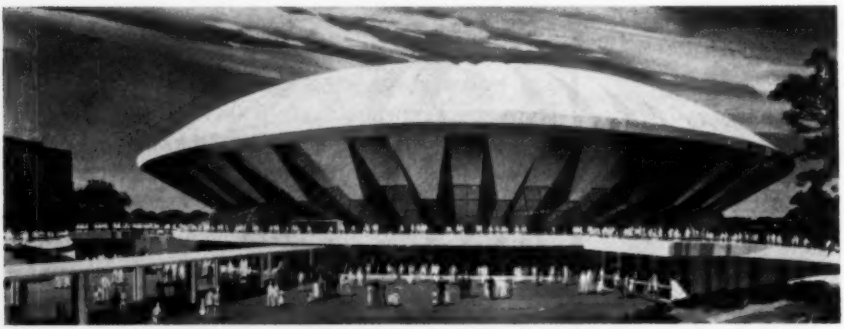
These, together with ACI publications and other important actions, are the primary official efforts toward attainment of the well-stated objectives of ACI. We all subscribe heartily to these objectives or we wouldn't be members; but what are we doing *as individuals* to foster and achieve these objectives?

I would like to suggest that each month, as our splendid JOURNAL arrives, we each stop a few minutes and ask ourselves what we are doing in our own sphere toward getting better concrete work. We might consider, for instance, depending on our relation to the work, such questions as: Can our specifications be made more specific with advantage as far as certain materials and practices are concerned? Are we trusting to luck on inspection or can we do more to put it on a dignified well-supported, and effective basis, unrelated financially to the contractor? Could we be advantageously more discriminating in the choice of aggregates and require practices insuring greater uniformity as

batched? Are we using the most efficient amount of the most favorable types of cement for the service conditions of our structure? Have we taken the opportunity to learn what many of the established reliable admixtures can do to improve our concrete? Are we using air-entrainment to ensure longer life in severe climates? Is it possible that a good pozzolan would add something to our concrete that cannot be otherwise as well obtained? Is there something more we can do to require and get more uniform mixes, the correct slump, and better workmanship in forming, placing and finishing? Would not more care in curing be worthwhile?

These and many other questions will suggest ways, with potentially beneficial results ranging from large to small, which will create a significant over-all movement toward attaining the ACI objective of better concrete work, if each of us, each month as our JOURNAL arrives as a reminder, *does something about it.*

Lewis H. Tuthill
President



NEXT YEAR THE UNIVERSITY OF ILLINOIS will have a new tradition-breaking indoor stadium. The stadium will be the first of its kind and will essentially comprise an inverted truncated-cone covered by a dome. No pillars or columns will be required to support the dome.

It will be supported by a 400 ft diameter post-tensioned concrete beam integral with the upper edge of the truncated cone. The outer circumference of the beam will be wrapped with 600 miles of prestressing wire to enable the relatively light structural member to carry the dome. The beam will have a cross-sectional area of approximately 20 sq ft.

The spectator stands will be erected in stepped fashion directly on the inside surface of the conical structure. A 36 ft wide deck will circumscribe the outside perimeter of the concrete cone approximately 12 ft from the ground. Spectators will enter the stadium through entrances at this elevated level, as well as through ground level approaches.

The deck will be largely enclosed in glass, as well the space beneath it. A glass wall will slope upward from the deck to the stadium wall, and will project downward to the ground to enclose the space beneath the deck.

A special wire winding machine was developed for prestressing the edge beam. The self-propelled machine will have two wheels which will ride the top surface of the dome-support ring, and two which will ride the outer face. The machine will be guyed to a point at the center of the stadium and will travel around the ring 2500 times, wrapping layer after layer of high tensile steel wire. The wire will be drawn, as it is wrapped through a die from 0.236 to 0.208 in. in diameter.

The prestressing operation will apply an initial inward force of 63,700 lb per ft to enable the ring to take the thrust of the dome.

The new hall will cost approximately $7,750,000. Architects are Harrison & Abramovitz, New York; Structural Engineers, Ammann and Whitney, New York; General Contractor, Felmley-Dickerson Co., Urbana, Ill.; Prestressing by The Preload Co., Inc., New York.

'61 Officers

Tuthill installed as ACI president

LEWIS H. TUTHILL, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento, was installed as ACI president at the 57th annual convention of the Institute held at the Chase-Park Plaza Hotels, February 20-23. He succeeds Joe W. Kelly, professor of civil engineering, University of California, Berkeley. Mr. Tuthill becomes ACI's chief executive after 2 years as vice-president.

Joining ACI in 1926, Mr. Tuthill has participated in nearly every phase of Institute activity. He has served on the Board of Direction and the Technical Activities Committee, and has chairmanned Committee 604, Winter Concreting Methods; Committee 611, Inspection of Concrete; and Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete. During his chairmanship of these committees the following publications were issued: *ACI Manual of Concrete Inspection*, 4th Edition: "Recommended Practice for Winter Concreting (ACI 604-56);" and "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)."

He is currently a member of five technical committees: Committee 116, Nomenclature; Committee 207, Properties of Mass Concrete; Committee 609, Consolidation of Concrete; Committee 611, Inspection of Concrete; and Committee 621, Aggregates.

Well known to JOURNAL readers, Mr. Tuthill has been a frequent contributor of technical papers for many years. He has prepared or collaborated on about 15 papers for the JOURNAL. His paper, "Concrete Operations in the Concrete Ship Program" (ACI JOURNAL, January, 1945) earned him the ACI Construction Practice Award. In 1956 he and William A. Cordon were awarded the Wason Medal for the most meritorious paper of the year for their joint contribution, "Properties and Uses of Initially Retarded Concrete."

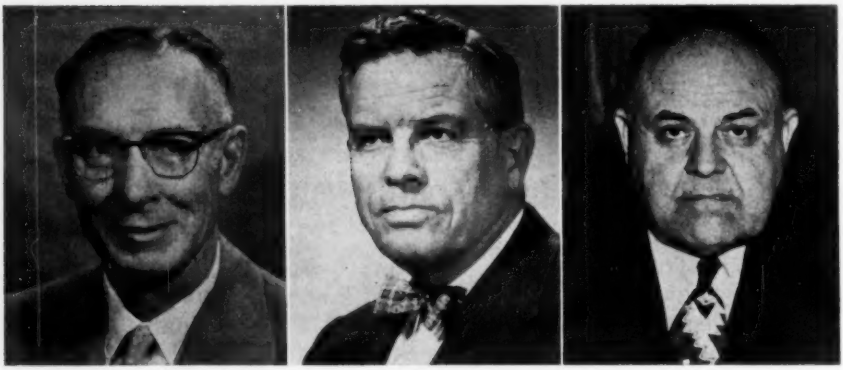
Mr. Tuthill assumed his present position in 1956. Previously, he had been associated with the Bureau of Reclamation in Denver for 17 years, and was chief of the Concrete Laboratory when he resigned to join the California State Department of Water Resources.

Since graduation from Oregon State College in 1920, Mr. Tuthill has been continuously engaged in design and construction work on irrigation, water supply projects, and the building of dams, with the technique and control of concrete as his primary interest.

Corbetta is new vice-president

Roger H. Corbetta, president, Corbetta Construction Co., Inc., New York and Chicago, was elected to a 2-year term as vice-president. An energetic supporter of ACI activities since joining in 1941, he represents the Institute on the Concrete Industry Board of New York City, an organization he helped found. He served as the CIB's chairman for 5 years.

Mr. Corbetta was a member of the ACI Building Committee, 1956-59, and was chairman of the local planning committee for the Institute's highly



ACI'S BIG THREE FOR '61 — (l. to r.) President Lewis H. Tuthill, Vice-President Raymond C. Reese, and Vice-President Roger H. Corbetta. Long active in furthering the aims of the Institute, the trio has a combined 70 years of membership with ACI.

successful 1960 convention in New York.

He has been identified with precast concrete structures since 1920 and pioneered in prestressing methods, having built a prestressed concrete barge for the U. S. Navy in 1942. He is one of the nation's leading concrete contractors.

Through his firm he has directed a diversity of projects in many parts of this country and abroad.

Reese continues as vice-president

Raymond C. Reese, principal of Raymond C. Reese & Associates, consulting engineers, Toledo, Ohio, begins his second year as an ACI vice-president. Mr. Reese was elected to a 2-year term in 1960.

Four directors installed

Four new directors were installed at the St. Louis meeting: Arthur R. Anderson, partner, Anderson, Birkeland and Anderson, structural engineers, Tacoma, Wash.; G. E. Burnett, assistant chief research engineer, U.S. Bureau of Reclamation, Denver; Chester P. Siess, professor of civil engineering, University of Illinois, Urbana; and Anton Tedesko, vice-president, Roberts and Schaefer Co., New York.

Dr. Anderson, an ACI member for the past 10 years, has specialized in static and dynamic strain measurements, stress analysis and prestressed construction. Since earning his doctorate from the Massachusetts Institute of Technology in 1938, he has amassed extensive experience in all these areas: as a research assistant and research associate at MIT: as a structural designer for the Aug. Klonne Steel Co., Dortmund, Germany; as technical director for the Cramp Shipbuilding Co., Philadelphia; and in private consulting and research work. He is currently a member of ACI Committee 324, Precast Reinforced Concrete, Thin Sections; and ACI-ASCE Committee 712, Precast Structural Concrete Design and Construction.

Mr. Burnett, assistant chief research engineer, U.S. Bureau of Reclamation, Denver, has been an ACI member since 1946. He graduated from the University of Utah in 1930 and has been employed in the Engineering Laboratories Branch of the USBR since 1936. Previously, he had worked in the refinery research laboratories of the Standard Oil Co. of New Jersey in Aruba, Netherland West Indies. At the present time he is a member of the Technical Activities Committee and

Arthur R. Anderson

chairman of the recently formed TAC Committee on Monographs. He is also serving on Committee 612, Recommended Practice for Curing Concrete, and Committee 616, Coatings for Concrete, both of which he formerly headed as chairman. Committee reports issued during his chairmanship include: "Recommended Practice for the Application of Portland Cement Paint to Concrete," "Guide for Painting Concrete," and "Curing Concrete."

Dr. Siess is a professor of civil engineering at the University of Illinois, Urbana, where he received his MS in 1939 and his PhD in 1948. Before joining the faculty at Illinois in 1941, Professor Siess worked with the Louisiana Highway Commission as a soils engineer, with the Chicago Department of Subways and Superhighways, and with the New York Central Railroad.

Much of Professor Siess' research has been published in technical journals. He has won many honors including, in collaboration with N. M. Newmark, the 1949 ACI Wason Medal for Most Meritorious Paper; the 1956 Concrete Reinforcing Steel Institute Award (with N. M. Newmark); and the 1956 ASCE Research Prize.

Active in committee work since joining ACI in 1938, Professor Siess currently heads Committee 115, Research; and ACI-ASCE Committee 321, Design of Reinforced Concrete Slabs. He is former chairman and now a member of ACI Committee 208, Bond Stress; former chairman of Committee 215, Fatigue of Concrete; and a member of ACI Committee 318, Standard Building Code.

Dr. Tedesko is a vice-president of the engineering firm of Roberts and Schaefer Co., New York and Chicago, and is located in the New York office. Educated in Austria, he holds the degree of civil engineer and Doctor of Science in engineering from the Institute of Technology of Vienna.

G. E. Burnett

Dr. Tedesko's name is identified with many wide-span structures and early prestressed concrete work in America. Among the first to advocate the use of industrial production methods in shell roof construction, he was directly responsible for the design and development of construction methods for a number of projects. A consultant to Headquarters, U.S. Air Force, he has recently been engaged in major basic programs for the Ballistic Missiles Division involving missile launching facilities.

He is a member of the Reinforced Concrete Research Council and is chairman of its Task Committee on Composite Construction. He is chairman of ACI Committee 334, Concrete Shell Structures, and a member of Committee 314, Rigid Frames for Buildings and Bridges, and Committee 315, Detailing Reinforced Concrete Structures. He has been an ACI member since 1946.

Mather appointed to fill vacancy

Bryant Mather was appointed by the Board of Direction to fill the vacancy on the Institute Board created by the election of Roger H. Corbetta to the vice-presidency.

Mr. Mather, supervisory civil engineer, concrete research, U. S. Army

Chester P. Siess

Anton Tedesko

Columbia Calcium Chloride, effective all year, is especially beneficial in concrete when temperatures drop below 70°.

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Engineer Waterways Experiment Station, Corps of Engineers, Jackson, Miss., is chief of the Special Investigations Branch, Concrete Division. Elected to the ACI Board of Direction in 1958, Mr. Mather had just completed a 3-year term.

Active in ACI committee activities since 1942, he completed 10 years of service with the Technical Activities Committee in February. He was TAC chairman from 1954 to 1956. He currently is working on Committee 116, Nomenclature; Committee 212, Admixitures; and Committee 612, Recommended Practice for Curing Concrete.

Mr. Mather received his degree in geology from John Hopkins University in 1936, and did graduate work there and at American University, Washington, D.C., until joining the Corps of Engineers in 1941.

Continuing board members

Continuing on the ACI Board of Direction are: A. Allan Bates, vice-president, Portland Cement Association, Chicago, Ill.; George C. Ernst, partner, Dunmire and Ernst, Lincoln, Neb.; E. A. Finney, director, Research Laboratory Division, Michigan State Highway Department, East Lansing, Mich.; Bruce E. Foster, materials engineer, National Bureau of Standards, Washington, D.C.; Ben C. Gerwick, Jr., president, Ben C. Gerwick, Inc., San Francisco; James A. McCarthy, professor, Department of Civil Engineering, University of Notre Dame, Notre Dame, Ind.; and Walter J. McCoy, director of research, Lehigh Portland Cement Co., Allentown, Pa.

The three immediate past presidents comprise the balance of the Board of Direction: Douglas McHenry, director of development, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.; Phil M. Ferguson, professor, Department of Civil Engineering, University of Texas, Austin; and Joe W. Kelly, professor of civil engineering, University of California, Berkeley.

Nominating Committee for '61

From 20 candidates nominated, five have been elected to the 1961 Nominating Committee: Harold Allen, chief, Physical Research Branch, Bureau of Public Roads, Washington, D.C.; D. L. Bloem, associate director of engineering, National Sand and Gravel Association and National Ready-Mixed Concrete Association, Washington, D.C.; Clyde E. Kesler, professor, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana; Thomas B. Kennedy, chief, Concrete Division, Waterways Experiment Station, Jackson, Miss.; and I. L. Tyler, research counselor, Research and Development Division, Research and Development Laboratories, Portland Cement Association, Skokie, Ill. Also on the committee are the three past-president members of the Board of Direction, Messrs. McHenry, Ferguson, and Kelly.

1962

First call for technical papers for presentation at ACI's 58th annual meeting, March 12-15, Denver.

Those who are interested in participating in the program should write to Institute headquarters before July 1, furnishing a synopsis giving the scope of the proposed paper and indicating features that the author thinks will justify its inclusion in the technical program.

Preliminary drafts should reach TAC for approval and acceptance by September 15; final manuscripts will be due by Jan. 1, 1962.

ACI Annual Awards

FIVE PROMINENT ENGINEERS were honored at the Awards Luncheon on February 22, when Joe W. Kelly, retiring president, presented the ACI annual awards on behalf of the Board of Direction of the Institute.

Turner Medal

The Turner Medal was awarded to Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready-Mixed Concrete Association, Washington, D.C., "for his many contributions to the basic knowledge of concrete through published literature and active participation in the work of technical committees of the Institute." The gold medal award was established in 1927 by the late Henry C. Turner, a past president of ACI.

Stanton Walker

Mr. Walker, an ACI past president (1947) and an honorary member (1958), has authored numerous papers dealing with concrete and mineral aggregates, 15 of which have appeared in the ACI JOURNAL. Since joining ACI in 1937, he has served on many technical and administrative committees. During his chairmanship of Committee 605, Hot Weather Concreting, the ACI Standard, "Recommended Practice for Hot Weather Concreting (ACI 605-59)," was developed. He is currently serving on Committee 613, Recommended Practice for Proportioning Concrete Mixes, and Committee 621, Aggregates.

Mr. Walker is a native of Indiana and an alumnus of the University of Illinois, Urbana. Following graduation he joined the research staff of the Portland Cement Association in 1917 and remained there until 1926 when he assumed his present position with NSGA. In 1930 he also became engineering director for the NRMCA.

Among other honors conferred on Mr. Walker are the Frank E. Richart Award, presented by the American Society for Testing Materials in 1957 for outstanding service; and two other ASTM awards — the 1951 Award of Merit and the 1960 Sanford E. Thompson Award. The Highway Research Board gave him the Roy W. Crum Award in 1956 for distinguished service. The Stanton Walker Fellowship was established at the University of Maryland in his honor.

In addition to his membership in ACI and ASTM, Mr. Walker is affiliated with the Highway Research Board and the American Institute of Mining and Metallurgical Engineers. He is a director of the Joint Research Foundation of NSGA and NRMCA at the University of Maryland.

Lindau Award

As recipient of the Lindau Award, Anton Tedesko, vice-president, Roberts & Schaefer Co., New York, was cited for "his numerous and outstanding contributions to the development and use of long-span concrete structures as exemplified by the thin-shelled arch."

The Lindau Award was founded in 1947 by the Concrete Reinforcing Steel Institute to honor the memory of Alfred E. Lindau, a past president of ACI. It is bestowed only for outstanding contributions to reinforced concrete design practice and is not restricted to ACI members.

Dr. Tedesko, newly-elected ACI director, studied engineering in Austria at the Institute of Technology, Univer-

sity of Vienna, where he received his civil engineering degree and later the degree of Doctor of Engineering. He also holds an engineering diploma from the University of Berlin.

An early exponent of industrial production methods in shell roof construction, he has been directly responsible for the design and development of more than 60 major shell projects, including the roof structure of the Lambert-St. Louis Municipal Airport in St. Louis.

He joined Roberts and Schaefer Co. in 1932 and became a vice-president in the New York office in 1956. Since 1955, he has also been a consultant to Headquarters, U.S. Air Force.

Dr. Tedesko is a member of the Reinforced Concrete Research Council and is chairman of its Task Committee on Composite Construction. He is also affiliated with the International Association of Bridge and Structural Engineers, and is a former director of the Illinois section, American Society of Civil Engineers.

An ACI member since 1946, he is chairman ACI Committee 334, Concrete Shell Structures, and is a member of Committee 314, Rigid Frames for Buildings and Bridges, and Committee 315, Detailing Reinforced Concrete Structures.

Kennedy Award

The Kennedy Award was presented to A. Allan Bates, vice-president, Portland Cement Association, Chicago, Ill., "for untiring efforts as chairman and member of the ACI Building Committee, successfully culminated in the outstanding architectural masterpiece housing the Institute headquarters." The modern, all-concrete building in Detroit was built in 1958.

The Kennedy Award, in the form of a framed scroll, was established in 1958 in memory of the late Henry L. Kennedy, a past president of ACI.

Dr. Bates joined the Portland Cement Association in 1946. Prior to joining PCA, Dr. Bates was manager of the chemical, metallurgical, and ceramic research division of Westinghouse Electric Corp.

A. Allan Bates

Dr. Bates has had broad international experience in industrial research. He spent much of the summer of 1960 touring the Soviet Union as chairman of a delegation of cement, concrete, and construction technologists. In 1957 he accompanied another delegation to a Pan-American Conference in Rio de Janeiro, Brazil, to discuss the possibility of coordinating engineering materials standards among nations of the western hemisphere.

Dr. Bates holds a BA in chemistry from Ohio Wesleyan University, Delaware, Ohio, and BS and MS degrees in metallurgical engineering from the Case Institute of Technology, Cleveland. His doctor of science degree "magna cum laude" was conferred by the University of Nancy, France. He also holds honorary degrees of Doctor of Engineering from Stevens Institute of Technology, Hoboken, N.J., and from Rose Polytechnic Institute, Terre Haute, Ind.

Currently, Dr. Bates is serving on the ACI Board of Direction. He is also president of the American Society for Testing Materials and is a member of the executive committee of the Building Research Advisory Board of the National Academy of Sciences.

Other membership affiliations include the American Society of Civil

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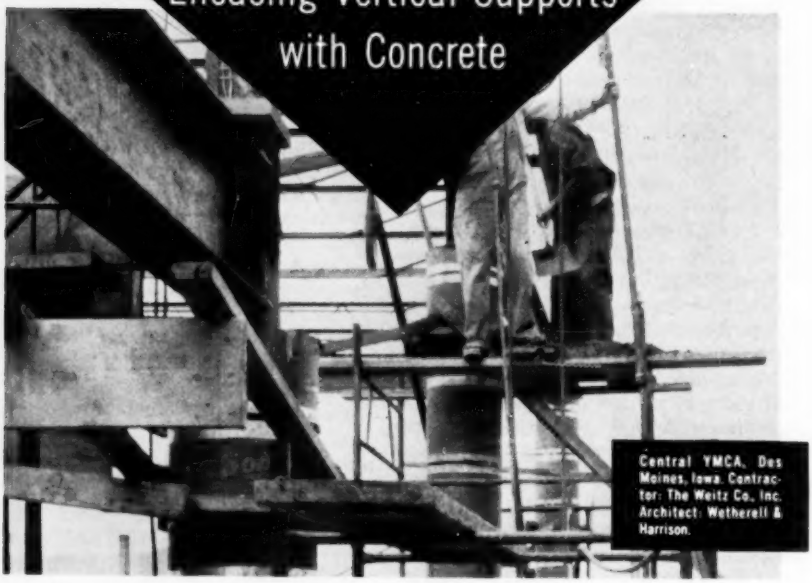

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Engineers, American Institute of Mining, Metallurgical and Petroleum Engineers, American Chemical Society, American Ceramic Society, American Society of Metals, and Western Society of Engineers.

Wason Medal for Research

Henry Toennies, assistant director of engineering, National Concrete Masonry Association, Washington, D. C., was awarded the Wason Medal for Research for his paper, "Artificial Carbonation of Concrete Masonry Units," published in the February, 1960, ACI JOURNAL.

Henry Toennies

Established in 1917 by the late Leonard C. Wason, the Wason Medal rewards the ACI member making the most noteworthy research reported in an Institute paper during the publication year.

Mr. Toennies is a graduate of Washington University, St. Louis, having received his BS in civil engineering in 1952. Following graduation he worked as a materials engineer for the Illinois Division of Highways. His next move was to A. M. Kinney, Inc., Cincinnati architects and engineers, where, as assistant laboratory chief, he field tested structural concrete aggregates, soils, and asphaltic mixtures.

He joined the NCMA in Chicago in 1954. With the transfer of NCMA headquarters to Washington in 1959, Mr. Toennies' responsibilities have included the establishment of a testing laboratory for research investigations on concrete block. His principal research interests have included laboratory and field investigations of new manufacturing techniques to improve physical properties of concrete block, and laboratory research into methods of testing masonry units.

In addition to his award-winning ACI paper, Mr. Toennies' research writings have appeared in other concrete journals.

An ACI member since 1958, he is currently serving on Committee 331, Structures of Concrete Masonry Units, and is chairman of the concrete masonry subcommittee of Committee 717, Practice in Low-Pressure Steam Curing.

Wason Medal for Most Meritorious Paper

The Wason Medal for Most Meritorious Paper was awarded to Robert A. Williamson, chief structural engineer, Holmes and Narver, Inc. Los Angeles, for his paper, "Performance and Design of Special Purpose Blast Resistant Structures," which was published in the May, 1960, ACI JOURNAL. This award was also established by Mr. Wason in 1917.

Robert A. Williamson

Since 1950 when he joined the Los Angeles engineering firm of Holmes & Narver, Mr. Williamson has had design responsibility for many of the blast resistant structures used in the Atomic Energy Commission's tests at the Eniwetok Proving Ground. In 1956 and 1957, he evaluated several types of buried shelters tested in Operation Plumbbob at the Nevada Test Site. In this operation he also served as a project officer in connection with other tests sponsored by the Civil Effects Test Group of the Nevada Test Organization. From 1956 to 1958, he made structural studies and reviews in connection with Air Force "hardened" missile base facilities.

Other experience includes 3 years in the evaluation of explosion and earthquake hazards relating to nuclear reactor facilities; 8 years in structural design of commercial, industrial, and military projects; and several years in aircraft structural design, and in of-

fice and field work pertaining to highway construction and surveys.

Mr. Williamson received his BS degree "with honors" in civil engineering from the University of California, Berkeley. A member of ACI since 1951, he is also affiliated with the American Society of Civil Engineers and the Structural Engineers Association of Southern California.

Treval C. Powers elected to honorary membership

HONORARY MEMBERSHIP IN THE AMERICAN CONCRETE INSTITUTE was conferred on Treval C. Powers, research counselor, Research and Development Division, Portland Cement Association, Skokie, Ill., at the Award's Luncheon, February 22, during the Institute's 57th annual convention at the Chase-Park Hotels, St. Louis.

Mr. Powers becomes the 32nd person so honored during the 55-year history of ACI. His many contributions, which have led to a better understanding of the chemical and physical phenomena underlying the formation of concrete from its component materials, were cited as the basis for Mr. Powers' commendation by ACI.

Treval C. Powers

Mr. Powers graduated from Willamette University, Salem, Ore., in 1925. He worked as a chemist and testing engineer for the Oregon State Highway Commission until 1927. Later he supervised concrete proportioning, inspection, and tests at Bull Run Dam, Portland, Ore. The special studies he undertook on this project are regarded as an important step forward in the field of concrete technology and were reported in his paper, "Concrete Studies at the Bull Run Dam," in the 1929 *Proceedings* of the ACI.

In 1930 he joined the PCA Research Laboratories where he was instrumental in the creation of a program of basic research. He was manager of the Basic Research Section from 1940 to mid-1958 when he assumed his present position as research counselor.

Mr. Powers was a 3-time winner of the ACI Wason Medal for Research: for work reported in his 1932 paper, "Studies of Workability of Concrete," for research described in a 1939 paper, "The Bleeding of Cement Paste, Mortar, and Concrete," and for work coauthored with T. L. Brownyard in their 1946-47 series, "Studies of the Physical Properties of Hardened Portland Cement Paste." In 1957 the American Society for Testing Materials presented him the Sanford E. Thompson Award in recognition of his notable work in the field of concrete and concrete aggregates.

Described in the ACI citation as "a rare combination of engineer and scientist," Mr. Powers has had more than 30 research papers published in concrete journals. He is also active in international symposia concerned with concrete technology. Most recently, he was cochairman of the program committee for the Fourth International Symposium on the Chemistry of Cement and Concrete.

In addition to his ACI affiliation, Mr. Powers is a member of ASTM, the American Chemical Society, and RILEM (International Union of Testing and Research Laboratories for Materials and Structures).

Committee 115 Research Compilation

ACI Committee 115, Research, Chester P. Siess, chairman, has published its 1961 compilation of research in progress on plain and reinforced concrete. This annual report is a valuable tool to researchers who have need of information pertaining to the work of other organizations, and can be helpful in avoiding duplication of effort.

The present compilation, assembled by the secretary of the committee, S. J. Chamberlin, professor of T & AM, Iowa State University, Ames, is divided into three sections, work underway at (1) United States academic institutions, (2) United States nonacademic institutions, (3) institutions outside the United States. There were 121 organizations reporting 803 projects this year. The number of organizations reporting in the three categories were: 45 United States academic institutions, 33 United States nonacademic institutions, and 43 institutions outside the United States.

Organizations in the following nations, in addition to the U. S., reported research projects this year: Australia, Canada, Denmark, England, France, Germany, India, Italy, Japan, New Zealand, Norway, Portugal, Scotland, South Africa, Sweden, Switzerland, and Wales.

The 36-page, mimeographed report contains a subject index. The projects are listed for each organization as are the active research personnel at the organization. The institutions are listed alphabetically within their groups; research outside the United States is listed under the country as well.

The committee is anxious to include all possible concrete research in this yearly listing; anyone having suggestions or knowing of omissions in the 1961 compilation should advise the secretary of Committee 115. Information will not be given publicity unless permitted by the correspondent. This information should be addressed to:

S. J. Chamberlin
Secretary, ACI Committee 115
T & AM Laboratory
Iowa State University
Ames, Iowa

Copies of the 1961 Compilation are available on request from Institute headquarters; address requests to:

Publication Sales Department
American Concrete Institute
P.O. Box 4754, Redford Station
Detroit 13, Michigan

Committee 115 was organized in 1921 and has been one of the Institute's most active committees since that time. The committee was formed to ". . . affiliate the interests of research workers and agencies in the field of concrete. Its assignment is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives." In fulfillment of this mission the committee produces the yearly Research Compilation and sponsors and directs the annual research session at the ACI conventions. These sessions are a highlight of the yearly meeting and consistently draw large audiences and interest.

The committee has a membership of about 90 persons representing an outstanding cross section of the membership both in the United States and abroad. The committee roster contains some of the most famous names in concrete research.

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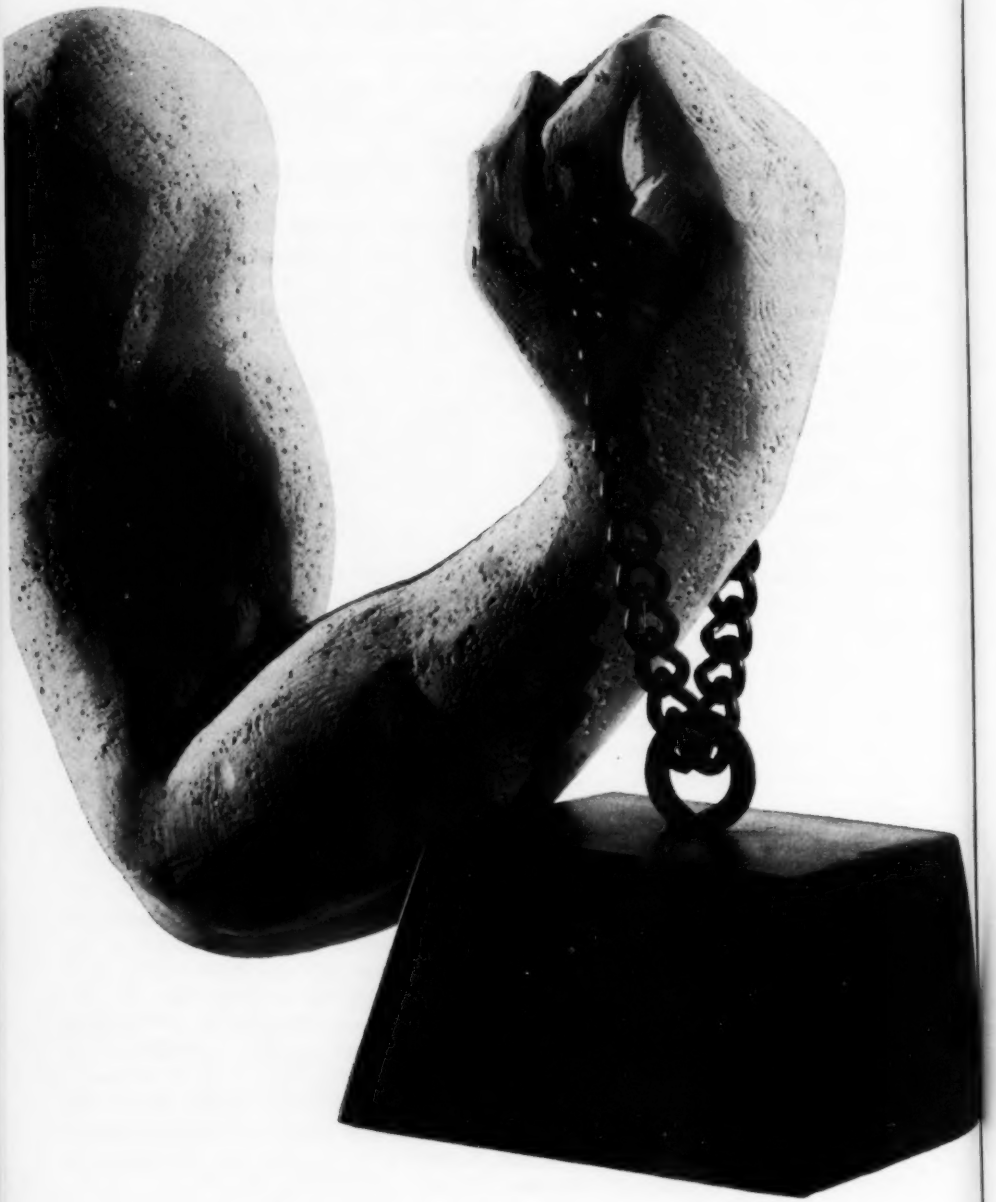

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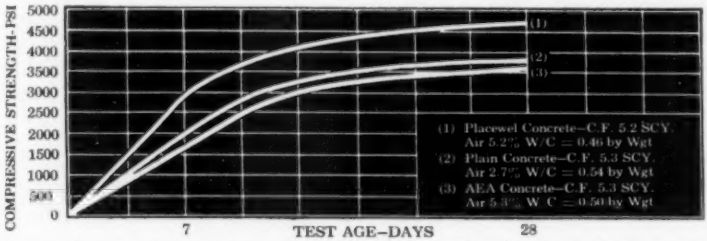

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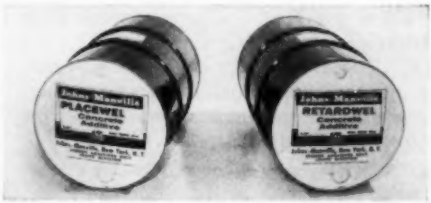

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What others are doing

Concrete Internationale

As further evidence of the interest of ACI members in what is happening beyond the "American" in American Concrete Institute, a growing number of them are participating in the meetings of overseas organizations; and an increasing interest in what is being done abroad is expressed by all ACI members.

In the past year, prominent ACI members traveled to Europe for meetings of the International Association for Bridge and Structural Engineering (IABSE), the International Bureau of Prefabricated Concrete (BIBM), the International Association of Testing and Research Laboratories for Materials and Structures (RILEM), and the European Concrete Committee (CEB). A delegation of Americans which toured the Soviet Union, and in turn hosted a Soviet group in this country, included many ACI members. ACI members were also found in attendance and on the program of the Fourth Symposium on the Chemistry of Cement which brought many well-known overseas engineers and scientists to the United States last year.

To help in keeping their fellow ACI members abreast of the activity in the concrete industry abroad, several of the participants in the various activities have written reports on the meetings attended. All but the 1961 meeting of the Comité Européen Béton (CEB) in Monaco, are presented here. The report of the ACI-CEB Collaboration Committee will appear in a later News Letter. Several of the reports were presented by their authors at the 57th annual convention in St. Louis.

SIXTH CONGRESS OF THE INTERNATIONAL ASSOCIATION FOR BRIDGE AND STRUCTURAL ENGINEERS

By DOUGLAS McHENRY*

In 1926 a group of bridge engineers assembled in Zurich to discuss problems and developments of mutual interest. A similar conference was held in Vienna in 1928. These meetings were so successful that the participants determined that a permanent organization, international in scope, should be formed. So it was that the International Association for Bridge and Structural Engineering came into being in 1929.

The IABSE normally meets every fourth year, and the first five congresses were held in Paris, Berlin, Liége, Cambridge, and Lisbon. A joint meeting of IABSE and the American Society of Civil Engineers was held in New York in 1958. In June 1960 the sixth congress was convened in Stockholm. About 800 engineers were in attendance, representing 38 nations.

*Member American Concrete Institute, director of development, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

A nontechnical description of a rather typical European technical gathering may be of interest to ACI members.

The opening ceremony of the congress was an impressive occasion held in the Stockholm concert hall, with the participation of a symphony orchestra. Welcoming addresses were given by various dignitaries, including F. Stüssi, president of IABSE, who recognized the representation of ACI by reading a letter from President Joe W. Kelly, which contained the following statement: "The opportunity for a free international exchange of technical information which your organization has arranged is an outstanding contribution to the greater cooperation between peoples which is the aim of all men of goodwill today. May your conferences form the basis for continuing exchanges of engineering information over the coming years." That statement elicited spontaneous applause.

Other functions were held at such renowned spots as the Town Hall, the Stockholm Cathedral, and the Drottningholm Court Theatre. The technical meetings were held at the Royal Institute of Technology.

The method of handling the technical sessions differed somewhat from the system with which most of us in ACI are familiar. To begin with, three languages were used — French, German, and English. Thus even in Stockholm, Swedish was not used at the technical sessions. Each chair in the meeting hall was equipped with headphones for receiving the simultaneous translations. Another point of difference is that about half of the total number of papers which constitute the proceedings of the congress had been published in advance in a volume entitled "Preliminary Publication." These papers were not presented at the meetings. The theme of each session was discussed by a "general reporter," and these discussions were based in large part on the papers already published. The new papers were, in general, presented by their authors only in summary form. These will be published in a "final report." The full proceedings of the Congress will then occupy two good-size volumes. A tremendous amount of technical ground was covered — too much to permit any summary treatment at this time.

Sessions

The six session themes covered such topics as basis of structural design, connections in metal structures, prefabricated concrete structures, and others. Particular emphasis was placed on the newer or more advanced concepts which are leading to more effective utilization of structural materials. These include a more accurate understanding of the safety of structures; the effect of creep, shrinkage, impact, and fatigue on strength and stability; designing by electronic computers; methods of assembling prefabricated elements; model testing as a design aid; etc. In addition, some of the papers described the design or construction of unusual bridges or buildings.

Collaboration

The aim of IABSE is "to promote collaboration between engineers of different countries for the exchange of ideas, of theoretical and practical knowledge, and of the results of research in the structural field." This is accomplished through the congresses at 4-year intervals, together with publication of the congress proceedings, an annual volume of "Publications," and an annual "Bulletin." The association does not prepare codes or standards, nor does it have a committee structure comparable to that of ACI. About a year ago three Working Commissions were organized as follows: I, General Methods; II, Metal Structures; III, Concrete Structures. The Commission on Concrete Structures is composed of one representative from each of 12 countries; Georg Wästlund, Sweden, is the chairman, and the

writer is the United States representative.

The aim of achieving international collaboration among engineers was indeed accomplished — in part through the organized sessions, and in perhaps even greater measure by the continuous succession of informal personal contacts and discussions (which also play such an important part in our own conventions). Advantage was taken of the world-wide attendance to also schedule committee meetings of other organizations, such as RILEM and the Comité Européen du Béton. Furthermore, the congress followed almost immediately after the Third International Congress of the Precast Concrete Industry, also held in Stockholm. All told, the 2-weeks, which included the 4 days of IABSE technical meetings was one of intense stimulating, and profitable technical activity.

THIRD INTERNATIONAL CONGRESS OF THE PRECAST CONCRETE INDUSTRY

By ARSHAM AMIRIKIAN*

The Third International Congress of the Precast Concrete Industry, which took place in Stockholm, Sweden, last summer, has an interesting origin. In 1954, on the initiative of the Belgian Institute of Cement (Chambre Syndicale Belge des Agglomérés de Ciment), and with the aid and sponsorship of the Belgian Office of the European Productivity Agency, a meeting was held in Brussels, Belgium, under the name of First International Congress of Precast Concrete. The purpose of the meeting was to create an international forum for the presentation and discussion of the problems and products of the precast concrete industry. Some 240 delegates from 22 countries attended the congress.

In view of the success of the first congress, it was decided to create a permanent organization which would assure the continuation of the collaboration of the participating countries and undertake the sponsorship of future meetings. This resulted in the founding of the International Bureau of Prefabricated Concrete (Bureau International du Béton Manufacturé), with the initial affiliation of ten European countries. Since then, four additional countries have joined the Bureau. While the United States does not as yet hold official membership in the bureau, the number of those who have been attending the conventions, either individually or as company representatives, has increased steadily, as has the number of their contributions to the technical programs. The bureau, better known by its initials BIBM, is governed by a committee composed of a representative from each of the member countries and is administered through a secretariat in Brussels. The congresses are convened triennially and the details of the programs are arranged in cooperation with the local group of the host country.

Second Congress

The second congress was held in Wiesbaden, Germany, in 1957. It was organized by the West German concrete industry, in cooperation with BIBM, and with the aid and sponsorship of the European Productivity Agency. There were over 700 registrants, representing 24 countries — including five from behind the "iron curtain." Three days were devoted to the technical program, arranged in morning and afternoon sessions, with

*Member American Concrete Institute, special structures consultant, Bureau of Yards and Docks, Department of the Navy, Washington, D.C.

Fig.1—In foreground, prestressed arch assembly of T-shaped segments. In background, partial assembly of multistory framing, spiral staircase, and ornamental drum head and panel assembly

two papers and discussions on a main topic for each session. The subjects covered the production of precast concrete units for many applications, including buildings, roads, and piping, the use of lightweight aggregates and cellular concrete, application of prestress, automation, and other problems of fabrication.

As an added feature, an exhibit was displayed on the grounds adjacent to the auditorium. This was essentially a demonstration of casting machinery and handling equipment produced by West German manufacturers. Arrangements were also made for a number of inspection trips to production plants located in the vicinities of Wiesbaden and Frankfurt. Of special interest was the manufacture of prestressed slabs, concrete elements of lightweight-aggregates, and centrifugally-cast concrete pipes.

THIRD CONGRESS

Stockholm, Sweden, was the host city for the third congress, which took place last June under the organizational guidance of BIBM and its local member — the National Federation of Precast Concrete Manufacturers of Sweden. Delegates from 24 countries attended the congress held at the Sankt Erikshallen. Those coming from the United States totaled 31, which constituted a remarkable showing, particularly since the United States is not officially affiliated with BIBM. At that time, 14 countries held membership in BIBM. It was estimated that the precast concrete industries in those 14 countries employed some 150,000 men, and that total production there grossed nearly 5 billion Swedish crowns, or roughly $1 billion — both impressive figures.

Technical program

All-day sessions for 3 days covered a rather crowded schedule. The program included general reports, special lectures and papers, floor discussions, and, an innovation, group discussions. The general reports, of which there were five, each consisted of a group of papers devoted to a main subject. According to this European procedure of international technical meetings, important parts of contributions from the various participants to each preselected topic are combined into a single paper and presented as a general report on that subject. The one given this task is called "general reporter." He makes the main presentations, followed by some of the other contributors who give elaborating talks on their particular views.

After each of the first four general report sessions, the audience was divided into four separate language groups (English, French, German, and "Scandinavian"—the official languages of the congress) for discussing the session topic. The results of these group discussions were then presented, through their respective chairmen, to the ensuing general session for further discussion and conclusions. The following were the topics of the general

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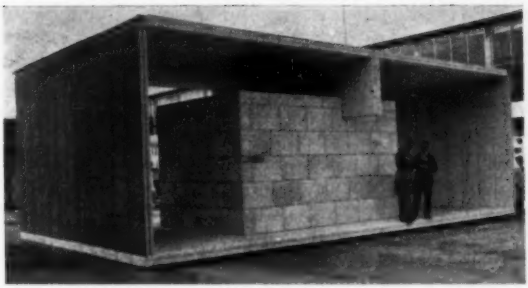

Fig. 2—Framing assembly, featured by tongue-and-groove planks of 20-in. modular width

report sessions:

1. Shrinkage of concrete
2. Large precast concrete units for buildings
3. Practical application of vibration to the manufacture of precast concrete products
4. Protection of concrete pipes against attack by aggressive soils and waters by the use of plastic coverings
5. Corporations' psycho-sociological problems in connection with their organization and internal human relations

The remaining sessions were devoted to miscellaneous topics of special application or production of precast concrete. These included bridge framings, building units, facing slabs, prestressed masts, and concrete block. There was also a paper on electro-curing of concrete products. Of a total of 26 papers on the program, six were contributed by the United States group.

Industrial show

Following the precedent set at the second congress in 1957, offerings of interest were augmented by an industrial exhibition. This was, however, a more elaborate show than the Wiesbaden display, with the emphasis on products rather than machinery. Briefly, it was a well-organized demonstration of the capabilities of the Swedish precast concrete industry, which has been active in this field for over half a century. According to the furnished statistical data, the volume of precast concrete work in Sweden during the past 30 years has increased tenfold, and that the present annual rate, excluding lightweight concrete and factory-made elements, exceeds 200 million Swedish crowns — about $40 million.

The most impressive part of the outdoor display was the framings of massive precast concrete members produced by the Swedish industry. These included: a bridge assembly, consisting of angular abutment walls and floor framing; a culvert frame of four large panels; a segment of multistory industrial building — featured by one-piece columns in three-story height; a gable frame — formed by tilting together two huge slabs; and an arch assembly — composed of T-shaped segments. Also of special interest were the framings of prefabricated houses — built of modular panels of cellular concrete and of light-

Fig. 3—Prefabricated house with large wall panels

weight aggregates, a variety of staircase segments, piping, railroad ties, ornamental and statuary pieces, and miscellaneous other castings. (Some of these are illustrated in Fig. 1-3.) The erection of some of the assemblies was a part of the demonstration.

The exhibit inside of the hall was featured by an array of photographs illustrating important examples of precast construction in western Europe. Machinery and equipment were also displayed, and some of the special processes and construction techniques were elaborated by motion pictures.

Social events

The Stockholm meeting had a full schedule of social events. In addition, there were a number of sight-seeing and inspection trips arranged. An added opportunity was provided for taking week-long tours of the country, both before and after the convention.

RILEM SYMPOSIUM ON STEAM CURED LIGHTWEIGHT CONCRETE

By JOHN K. SELDEN*

The RILEM Symposium on Steam Cured Lightweight Concrete was held at Gothenburg, Sweden, June 21 to 23, 1960, immediately following the close of the Third International Congress of Precast Concrete Industry. RILEM stands for International Association of Testing and Research Laboratories for Materials and Structures.

The Symposium, held at Chalmers University of Technology, drew more than 250 registrants, representing 24 countries, including the Congo, India, Japan, Canada, the United States, most of the European countries, Peoples Republic of China (mainland) and half a dozen from the Soviet Union.

President Joe W. Kelly's memo in the News Letter in February, which stressed ACI's "Internationality" and the beneficial influence of rubbing shoulders with technical people of other countries, was well illustrated here. Thanks to the good-natured rivalry between our host city and Stockholm, Gotenburg outdid itself in providing social functions for getting acquainted. The city invited us to a dinner and dance at the Yacht Club, The Harbor Authority took us for a boat ride and the Contractors' association entertained us at a formal Swedish-style banquet. This banquet provided an opportunity for welcoming speeches by a dozen different nationalities including J. J. Shideler for the U. S.

Technical contacts

The technical contacts also were profitable. For instance, Walter Albrecht of the Otto-Graf-Institute, Stuttgart, Germany, a pioneer in cellular concrete testing, told me of his forthcoming book on concrete, which I was able to secure as soon as published. There was no "iron curtain" in technical contacts — only a language barrier. Through the aid of an interpreter, I began discussing with Peter Bozjenov, head of the Soviet delegation, the relative merits of the two basic systems of producing cellular concrete. Siporex, Ytong and Durax, the Swedish systems, used powdered aluminum to generate gas voids, while the earlier Russian and the present American practice have both shown a preference for preformed foam. Recently gas expanded concrete has also become popular with the Soviets, but the two of us agreed on a preference for foam.

This international symposium on autoclaved cellular concretes was the

*Member American Concrete Institute, consultant, concrete and autoclave products, Toledo, Ohio, (formerly technical director, Autoclave Building Products Association).

first of its kind ever held. In the past, familiarity with many of the properties of this material, the bases of design, the precautions to be taken, and the new potentialities, were something of a trade secret with the Swedish promoters. Now, perhaps due in part to the fact that many of the basic patents have expired, every major phase of testing and design of autoclaved cellular concrete has been discussed and presented in a set of 50 preprinted papers of high technical quality. While a great deal more needs to be learned, aerated concrete can now, with the aid of this reference library, be considered a new branch of materials research and structural design where ever the material is on the market and able to compete economically.

Technical program

The three-day technical program was divided into major themes, one for each half day with the last afternoon given to a summary. Most of the General Reports were read first in English and then translated into as many other languages as requested. A majority of the preprinted papers were available in English, but only a few of the 50 papers were actually presented orally.

The symposium was limited to autoclaved lightweight concrete and to design, testing and construction problems, with no mention of manufacturing. One theme was deformation and strength properties of lightweight concrete. Main emphasis was placed on compressive strength which is directly related to bulk density and of great practical and economic significance. However, tensile strength, which largely governs cracking, and shear strength, so important in laminated or plaster panels, must both be studied. Curves presented indicate maximum strengths for autoclaved cellular concrete up to 1000 psi, at 32 lb per cu ft density and 1500 psi at 40 lb per cu ft density.

Experiments on storage conditions, after autoclaving, indicated that strengths increased 16 to 20 percent when stored in room air, or under water, but only 9 percent in a damp basement. In carbon dioxide-rich air over water, cellular concretes made with cement silica alone, did not lose strength, but samples made with lime, lost an average of 12 percent or more.

Not only is lightweight concrete more susceptible to chemical attack by CO_2 and by small traces of sulfur compounds in the air, due to its porosity, but it is also influenced by mechanical stresses like vibration which means field conditions may be more severe than protected laboratory tests. Hence an adequate factor of safety is necessary.

The ten excellent preprinted papers, which for this theme only, were in German, indicate the extensive research into physical, chemical and mechanical properties of cellular concrete which have been conducted in Europe.

Reinforced lightweight concrete

Another theme was reinforced lightweight concrete. In the past, cellular units have been designed by emperical methods, since, unlike ordinary concrete structures, the units are mass produced in a factory and can't be pretested to determine the strength with a given factor of safety. Now, however, with large production programs, where more than a thousand different shapes, sizes, and types of cellular concrete products are made, the calculation theory of design becomes necessary. In comparison with ordinary concrete it must be remembered that cellular concrete has lower compressive strengths, modulus of elasticity, bond strength, and density, and that during the autoclave process at temperatures of about 350 or 360 F there is a differential expansion between the steel and concrete. For this reason special attention should be given to anchoring the bars and to the calculation of internal stresses as well as those from loading. Attention must be given to the loss or elimination of bond due to (a) the autoclave process itself, and (b) the layer of bituminous

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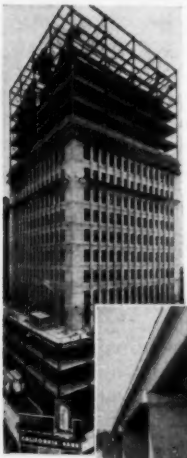

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type corrosion protection coating on the steel.

Because of this loss of bond, welded transfer bars may be used for anchorage. While the ultimate moment of a beam or slab may depend only on the yield point of the properly anchored steel, the flexural rigidity, which is important for roofing slabs, depends on the bond and percentage of compressive reinforcement.

Heat insulation

Heat insulation and migration of moisture was the theme of one of the sessions. The fact that ten separate scientific papers on this subject were presented, indicates its importance. Not only are damp walls undesirable from their appearance, the possibility of mold growths and damage to draperies, etc., and the effect on comfort, but also they promote corrosion and espcially in the case of cellular concrete, a marked increase in thermal conductivity. Thus taking a bone dry wall as 100 percent, the K value is increased 25 percent for cellular concrete under average conditions, with 5 to 6 percent moisture and 175 percent with rain-soaked walls of 35 percent moisture content. In the case of roof slabs, thermal conductivity may be two or three times as great on the cold side as on the warm, due to difference in moisture content. Of course, moisture from within and from without must both be considered.

Because of the importance of determining the moisture content in buildings already erected, a nondestructive test method has been developed by the British Building Research Station, using a portable radio transmitter and receiver which, on scanning the wall can identify the moist spots by the attenuation of the signals received.

Surface treatment

Surface treatment was discussed at one session. Cellular concrete walls are generally coated on the outside to protect them from the weather and on

the inside from impact damage and to reduce the dust collecting nature of the porous surface. The walls should be kept dry because moisture does the following things: heat insulation is impaired, strength reduced, susceptibility to frost damage increased, steel reinforcement and nails corrode unless properly protected, and efflorescence encouraged. The exterior surface must, therefore, be water repellent and porous enough to permit moisture evaporation, or else completely watertight such as a bituminous or paint coating with ventilated air spaces behind.

Cement or lime cement stuccos or renderings should not be over 1/16 or 1/8 in. thick because of possible damage to the cellular structure beneath. Stearates and silicons are widely used to give water repellency. These must be applied on uncracked surfaces. Recently attractive surface finishes made with silicate paints and polyvinyl acetate coatings, with fine marble or ceramic colored granules, have been introduced which give attractive surfaces. Cellular blocks may, of course, be used as back up for face brick, tile, etc.

CONCRETE CONSTRUCTION IN RUSSIA — 1960

By A. ALLAN BATES*

By agreement between the governments of the United States and the Soviet Union, the years 1960 and 1961 have been designated as a period of broad exchange of information in scientific and cultural fields between the two countries. Numerous specific areas of science have been chosen and the exchanges have been carried out through extended visits and tours of American scientific and technical delegations to the Soviet Union and return visits by similar Soviet delegations to the United States.

One of the technical areas of exchange concerned construction and, particularly, uses of hydraulic cements and concretes in construction. The American delegation went to the Soviet Union in the summer of 1960 and traveled freely and extensively, visiting many research institutions, cement and concrete plants, and construction sites in European Russia, as well as beyond the Ural Mountains.

Members of the American Concrete Institute participated in the American delegation to the Soviet Union and many other members of ACI served as hosts to the Soviet delegation in this country. The following brief paper is a report to the Institute on some aspects of the exchange.

The entire episode constitutes one of the freest and most extensive instances of scientific communication which has taken place between the two countries. On both sides there was a friendly welcome offered to the visiting delegation and technical discussion was subjected to no reservations or limitations within the designated area of interest. It seems that such an excellent start of understanding should be nourished and cultivated rather than allowed to wither. On the side of the United States, the American Concrete Institute is the natural agent for this purpose. Suggestions are made as to how it may fill this role.

The members of the American Concrete Institute must certainly find an interest in the prospect of a vast contemporary nation which, in effect, has decided, as a general policy, that its buildings of the future will be constructed of reinforced concrete.

One of the little discussed, but perhaps most important, aspects of the Soviet Union today is the tremendous

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Fig. 1—Standardization. Partial view of an apartment construction project, completely precast

campaign of urban construction which in the post World War II period has been rising in a steady crescendo to heights never previously reached in the history of any nation. The reasons for and consequences of this construction campaign may turn out to be almost as important to the rest of the world as they are to the Soviet Union. I shall therefore discuss briefly some nontechnical aspects of this Russian situation even though to an ACI audience the technical aspects will naturally be of principal interest.

Agricultural beginning

The revolutionary Soviet government took over a country which had been widely devastated by World War I and subsequent revolutions and counter-revolutions. Furthermore, the country for eight centuries had been held by absolutist rulers in a state of relatively backward economic development. The Renaissance which swept western Europe out of the medieval and into the modern era during the 13th to the 16th centuries A.D. was scarcely felt in Russia. Something akin to that Renaissance is now exploding in the Soviet Union with a speed and violence which may or may not have results similar to those of the more deliberate rebirth of western Europe. The western renaissance was a by-product of a new urbanization which brought into being and to power such cities as Florence, Genoa, Venice, and Paris. The great rebirth grew out of the economic, intellectual, and cultural communion among these cities.

Fig. 2—Precast sandwich panels with cast-in windows. In the shipping yard of an "apartment factory"

Until well into the 1920's, the Soviet Union was overwhelmingly an agricultural nation of rural people. Probably fewer than 15 percent of the population were city dwellers engaged in industry and commerce. The revolutionary government which took over at that time decided that the Soviet Union must become an industrialized nation. This is equivalent to the determination that it must become an urbanized nation. The great and pressing problem, then, was to build cities and move farm and country dwellers into them where they could be trained and formed into industrial forces. By 1960, the USSR has become a nation almost evenly divided in population between city and country. Calculating from the basis of Russian national population in 1925 and 1960, this means that some 50 million former farm dwellers have been provided with city homes. Furthermore,

over 25 million other persons whose homes were devastated by World War II have been provided with dwellings and other facilities and, finally, notable changes in standards of living among Soviet citizens of the last generation have added further construction tasks. In total, it seems that a number of persons approaching 100 million may have been provided in the Soviet Union in the last generation and a half with town and city dwellings and with most of the accompanying necessary facilities, such as factories, stores, streets, schools, etc. A large part of this has been accomplished in the last 10 years and the pace is still rising so that today much of the Soviet Union looks like a vast construction site.

Construction magnitude impressive

It must be pointed out, and knowledgable Russians are the first to admit this, that the standards and the quality of the resulting buildings have been generally mediocre and, in many cases, so low that only the dire Russian housing emergency has permitted their acceptance and use. Their structures have been improving rapidly in the last few years but they are still below the standards and the quality which would be required for most American purposes. Also intense crowding still persists in Russian urban communities and the amount of space available to the average Soviet family is far below that enjoyed by the average American family. Thus, both qualitatively and quantitatively, notable deficiencies are still evident. Nevertheless, the sheer volume of building which has been accomplished constitutes a construction achievement of unprecedented magnitude. To have done it at all has required firm, shrewd, long-range decisions and relentlessly organized and expedited activity which must be seen to be appreciated. Allowance should also be made for the fact that this building work has been carried out under immense pressure during a time

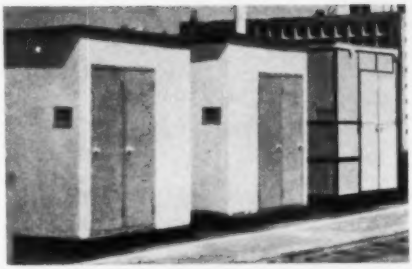

Fig. 3—Complete precast baths and other rooms ready for installation

of wars and intense troubles in Europe and in the Soviet Union.

The Soviet planners realized that they could not industrialize and urbanize their country without industrializing construction itself. Using traditional methods and materials, the time required to do the job would have been excessive. The circumstances which led or forced them to a decision to concentrate on the use of reinforced concrete included the shortage of skilled labor, lack of a well-developed national transportation system, necessity of conserving industrial energy, the amenability of reinforced concrete to industrial production methods and the relative availability of materials for concrete-making in many widely-scattered areas of the Soviet Union.

Mechanization introduced

By the summer of 1960, over 2000 factories were said to be on assembly-line production of precast concrete building units in the USSR and more were being built as rapidly as possible. Mechanization and automation have been introduced extensively into these Russian plants; to such an extent, indeed, that the high capital investment involved would be seriously questioned if resorted to in comparable factories in the United States. However, the productivity of these Russian precast concrete plants in terms of completed apartments, industrial buildings, transportation structures, etc., is most impressive. The factory production of

Fig. 4—New style apartments. Each unit completely precast in factory then stacked on building site to form the final structure. Model in foreground

apartments, especially, has been carried to remarkable extremes in recent years. Every component of the apartment building is precast, including foundations, walls, floors, roofs, stairways, and when used, beams, columns, and girders. Complete precast kitchens and baths with all service lines cast in place are delivered to building sites. Finally, in 1960, they began apartment construction in which complete individual flats, including kitchen, bath, and two or more rooms are precast and piled together in the necessary number to form a multiflat apartment building of any size proposed. With such procedures in use, the working crew at the building site may comprise little more than a crane man and a half dozen moderately skilled laborers to attach crane slings and connect fittings.

Bold techniques

This complete industrialization of construction is neither new in principle nor original with the Soviets. Nor do they claim that it is. Much the same may be seen in France, Germany, the Scandinavian countries, and elsewhere, usually producing better quality buildings than have been turned out in Russia. But the Soviets have shown great boldness in carrying out this industrialization to extreme limits in recent years. And they have exhibited ingenuity and high technical competence in the mechanization, automation, and streamlined layout of their precast concrete factories.

They have taken their climate into account in their building designs and practices. To assure the comfort of occupants, the Soviet concrete engineers have effectively developed insulating wall panels, using all means known elsewhere. Thus, sandwich panels with foamed concrete cores are a standard product. Likewise, they make extensive use of lightweight insulating aggregates, of porous concrete with open gap-graded aggregate mix, of double waffle-slab panels enclosing central insulating air zones, of concrete mixes incorporating wood fibers and other nonconducting organic substances. All of these concrete products are manufactured in quantity by industrialized methods developed largely in Soviet research institutes. Some of these methods and the equipment involved are quite unusual, for example, rolling mills which semicontinuously form waffle slabs up to 13 ft wide and 20 ft long.

Research

The research institutes in which equipment and processes are developed have complete responsibility not only for the inception and initial research on an idea but also for carrying it through to final industrial practice. This includes design and erection of such plants as well as their operation

for the period necessary to achieve satisfactory commercial success.

Thus, the Soviet cement and concrete research laboratories and staffs tend to be large, with a few running to a thousand or more workers in a single institute, many of whom are field engineers and technicians. This centralization and continuity of responsibility for scientific and technological development seems to have served the Soviet national purpose most effectively. A rather extended meaning of the word "research" arises from the nature of the Russian research institutes devoted to study of cement and concrete problems. But by this extended definition, or by any other, it must be admitted that the Soviet commitments to research and development are great indeed; certainly exceeding those to be found in the United States or any other nation.

This central position of research and development in the Soviet scheme of things is pervasive throughout the cement and concrete industries. The research institutes, which are, of course, government organizations, control or guide every activity from geological search for raw materials to design of finished structures. The intensive standardization of materials, designs, procedures and practices which results has been the necessary foundation of success to the Soviet building achievement. But the high efficiency has been gained at the cost of a monotony in urban appearance which is difficult to disguise.

Soviet cement and concrete science and technology are well advanced. Adding to their own extensive original studies, the Soviet scientists and engineers also remain continuously well informed about the work and progress of others throughout the world. They exhibit willingness and a desire to communicate and exchange ideas and information, much like their equals in western countries. The basic research work carried out in their institutes and in their universities is extensive, well planned, and competently done.

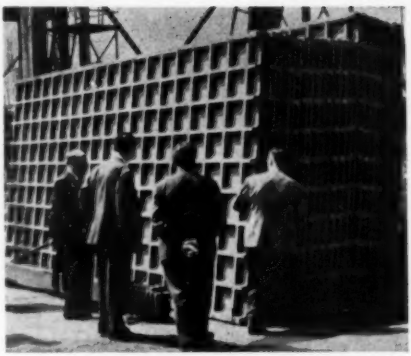

Fig. 5—Rolled reinforced concrete waffle slabs. In factory shipping yard

Applied research and development work are done on a large scale. Because of the powerful central position of the research institutes, results can be carried directly into practice with unusual speed. In consequence, change and progress in industrial and construction practices are not as hampered by the inertia of the immense and complicated Soviet planning organizations as many western observers think they must be. Progress seems particularly evident in the buildings of very recent years as compared with those erected only a few years earlier.

Quality concrete

The principles of high quality concrete making are well known and generally observed in the Russian precast plants. Dry mixes, high in cement content, heavily vibrated, tamped, pressed or rolled into the forms and subsequently steam-cured under automatically controlled cycles are all in prevalent use. Many of the plants use curing cycles notably higher in temperature and somewhat shorter in time than is generally permitted in the United States.

Natural aggregates of high quality are relatively scarce over wide areas of the vast, geologically undifferentiated Soviet steppe and prairie lands. Some of the aggregates now in use there would be rejected in the United States,

Fig. 6—Bridge girders. Precast, pre- and post-tensioned

but the Russian producers, driven by tremendous pressure of demand, compensate for unavoidable aggregate deficiencies by using high cement contents and other expensive but more or less effective procedures. The development and use of synthetic aggregates are vigorously pursued. Research work in this field has high priority.

Quality control in concrete plants receives reasonable attention and is generally of acceptable level although this is sometimes visibly weakened by local circumstances, such as poor aggregates, overextended methods, insufficiently skilled labor, and (as sometimes elsewhere) overwhelming pressure for production. Badly finished surfaces, edges, and corners of precast concrete structural members were still a surprisingly common sight in the USSR of 1960. At that same time much good concrete was also in evidence — so no entirely general conclusion can be drawn concerning quality in the Soviet Union any more than such conclusions can be drawn in the United States.

Cement plants

Cement plants in the USSR are essentially like those in the United States. With the exception of some major equipment still recently purchased abroad, all new and prospective plants are designed and built according to standards developed in one of several central research institutes. Productive capacity has been increasing at a great rate in recent years and continuing annual increases of some 7 to 8 million tons (35 to 40 million bbl) are planned for the years up to 1965. Nevertheless, demand continually exceeds supply and every expedient is used to extend the tonnage produced. Thus, additions of slags, pozzolans, reactive industrial waste products, fly ash, etc., to portland cement are almost universal. In fact, somewhat over half of the total cement tonnage produced is of portland-slag, portland-pozzolan, or other mixed type. By definition, even "portland cement" may contain at least 10 percent of such admixtures. Naturally it would seem that the use of these various cements in the precast concrete plants would require that careful attention be given to suitable and necessary variations in concrete mix proportions, curing cycles, etc., to accord with the characteristics of the cement used. However, the requirement is partly obviated by the fact that all cements are produced and purchased to standard strength performance specifications. Thus a Class 300 portland cement is presumed to provide the same strength in a given concrete as is a Class 300 portland-pozzolan cement. Nevertheless, quality control of cement under this system is neither so close nor so reliable as to make cement testing entirely unnecessary in concrete plants. Apparently local managerial discretion is required and permitted in this matter.

The Soviet Union is engaged in a gigantic revolutionary leap from a relatively backward agricultural-rural past to a hoped-for industrial-urban future of ultra-scientific efficiency. It is attempting this on the basis of continuous national planning operations on a scale and of a complexity never before attempted. The success of these plans rests in no small measure on the maximum possible use of reinforced concrete.

Technical institutes

If there had been an association like ACI in Russia, it would certainly now be playing a central and vital role

in the Soviet national scene. But an "ACI" which is a free association of individuals, organizations, and agencies banded together for mutual assistance and voluntary national service is not consonant with Soviet thinking and planning. They prefer an official federal and regional network of research institutes and technico-economic councils coordinated under state jurisdiction through an Academy of Construction and Architecture. This system constitutes a vast national "department of building technology" and it is evident to any objective foreign observer that, relative to Soviet national purposes and circumstances, the system is succeeding within reasonable expectation. For, even after taking some dubious features into account, the effective advancement of building technology remains broadly visible and undeniable in Russia.

To us in the United States, the tradition of free and voluntary association in national technical societies such as ACI is a cherished principle. It has also been an effective means of progress relative to American purposes and circumstances. It seems certain that the two systems for technical progress, the Russian and the American, will continue indefinitely in their separate and different ways. But this should not mean that the two must remain largely out of contact with each other. ACI has become a strongly international body. It must be that we have much to give to our colleagues in the concrete and construction industries of other lands — for they join us willingly. It is also abundantly evident that ACI gains richly from its members in Canada, Latin America, Europe, Africa, and Asia. Almost every major advance in the historical progress of concrete science and technology has had its first roots in these areas outside of our boundaries. ACI serves its home country to great purpose by acting as a native yet international intermediary for bringing into common availability and use the results of progress made everywhere in the realm of concrete technology and construction.

Role for ACI

I suggest that we of the American Concrete Institute give special consideration in the coming year to the possibility of extending and continuing the communion of interests between Soviet and American "men of concrete" which was established so amicably and effectively in 1960. The tremendous Russian commitment to research and development in cement and concrete technology is already bearing early and interesting results. The output of important ideas and information from their laboratories is certain to grow impressively in the future. It can only be to the common advantage of all concerned to add their contributions to the fund of information shared by all. The Russian cement and concrete scientists and engineers showed their desire to do this by their liberal and friendly participation in the Soviet-American exchange of 1960.

Because of the restraints on contact and communication which have persisted between the USSR and the USA, any plan or proposal for action by the American Concrete Institute toward their improvement would have to be made through carefully selected channels. Due consideration will have to be given to all interests concerned, both governmental and private. The respective academies of science of the two nations, through their branches which deal with building research and technology, construction, and architecture, would probably be interested and helpfully concerned. In any case, through whatever channels or by whatever means contacts may be established, I believe that the American Concrete Institute is here uniquely qualified to initiate, propose, and follow a service opportunity of great potential value to the world. The work of building is a peaceful and constructive activity in every country. The Soviet architects and builders live in a technical realm

of reinforced concrete. ACI is the largest and most diversified international society for the advancement of reinforced concrete.

Reinforced concrete may provide the foundation for a continual and mutually advantageous meeting of the two great nations. May not we of the American Concrete Institute take a helpful and considered initiative?

The first steps should be modest and may result in only modest progress. Invitations and offers for contribution of papers, for participation in meetings, for exchange of visits and lectures — these and other customary forms of contact and communication suggest themselves.

I expect to propose that our ACI Board of Directors give thought to these matters. I hope that members of the Institute agree that the purposes are worthy of the effort.

FOURTH INTERNATIONAL SYMPOSIUM ON THE CHEMISTRY OF CONCRETE

The Fourth International Symposium on the Chemistry of Cement was held in the United States, October 2-12, 1960 under the joint sponsorship of the National Bureau of Standards, representing the United States government, and the Portland Cement Association in behalf of the cement industry in the United States and Canada. Over 250 chemists, physicists and engineers, representing 35 countries, convened in Washington, D.C., Oct. 3-7, for the formal presentations of the symposium. About one-half of the participants were from the United States.

Following the formal sessions, the overseas visitors motored cross country by bus to Skokie, Ill. The 2-day trip provided the opportunity for sightseeing and observing highway systems and their use of concrete. At Skokie, 3 days, October 10-13, were spent in seminars, discussions, and studying projects of the PCA Research and Development Laboratories.

Washington session

The technical program at Washington, D.C., was divided into seven major parts which covered:

- Chemistry of clinker
- Chemistry of hydration of cement compounds
- Chemistry of hydration of Portland cement.
- Properties of cement paste and concrete
- Destructive processes in concrete
- Chemical addition and admixtures
- Special cements

Each of the major parts was allocated one-half day. Papers, supplementary papers, and written discussion were submitted in advance and copies provided to all participants.

All written and oral presentations were in English. The presentation of papers was limited to 15 min and supplementary papers and written discussions were limited to 10 min each. General discussion from the floor was permitted after presentation of the written contributions.

At the first session A. T. McPherson, associate director, National Bureau of Standards, and G. Donald Kennedy, president, Portland Cement Association welcomed the participants to the symposium. Eberhard Spohn, president, Heidelberg Cement Co., West Germany, responded in behalf of the participants. Wallace G. Brode, former associate director, National Bureau of Standards, and foreign science advisor to the Department of State, discussed "Some Problems Associated with the Growth of Science" and F. M. Lea, director, Building Research Station, United Kingdom, presented "Cement Research — Retrospect and Prospect."

Chemistry of clinker

The second session was devoted to the chemistry of portland cement clinker. Henri Lafuma, director, Central Laboratory for Research on Hydraulic Cements, France, and R. H. Bogue, consultant, United States, presided as cochairmen. The principal subjects discussed were: phase equilibria and constitution of portland cement clinker, crystal structure of clinker constituents, solid solutions of the minerals of portland cement clinker, the effect of minor components on the hydraulicity of the calcium silicates and the reactions of coal ash with portland cement clinker during the burning process.

Chemistry of hydration

The third session provided a discussion of the chemistry of hydration of cement compounds. Fritz Keil, director, Research Laboratories of the Cement Industry, West Germany, and Charles E. Wuerpel, vice-president, Marquette Cement Manufacturing Co., United States, presided as cochairmen. Among the subjects discussed were: the hydration of tricalcium silicate and betadicalcium silicate at room temperature, hydrothermal reactions and the steam curing of cement and cement-silica products, the hydration of calcium aluminates and ferrites, and the influence of high pressure steam on the process of hardening of calcium and magnesium silicates and of cements of different compositions.

Session four was devoted to a discussion of the chemistry of hydration of portland cement. Niko Stutterheim, vice-president, South African Council for Scientific and Industrial Research, and Myron A. Swayze, director of research, Lone Star Cement Corp., United States, presided as cochairmen. The principal subjects of discussion were: early hydration reactions of portland cement, false set in portland cement, the chemistry of hydration of portland cement, and procedures for studying the hydration of cement.

Cement paste

Wolfgang Czernin, director, Research Institute of the Association of Austrian Cement Manufacturers, and Hubert Woods, director of research, Portland Cement Association, United States, presided as cochairmen of session five which was devoted to a discussion of the properties of cement paste and concrete. The major topics discussed were: physical properties of cement paste, the microstructure of hardened paste, and volume changes and creep of concrete.

Session six was a discussion of destructive processes in concrete. Arturo Rio, director, Research Center of Cement Manufacturers, Italy, and Douglas Parsons, chief, Building Technology Division, U.S. National Bureau of Standards acted as cochairmen. The subjects discussed were: chemical reactions involving aggregates, frost action in concrete, and the resistance of cement pastes to aggressive media.

Session seven consisted of one paper on some chemical additions and admixtures in cement paste and concrete by H. E. Vivian, Australia.

B. G. Skramtaev, member of the Presidium of the USSR Academy of Construction and Architecture, and A. R. Collins, director of research and technical service, Cement and Concrete Association, United Kingdom, were cochairmen of session eight which was devoted to a discussion of special cements.

All written contributions to the Symposium will be published by the Nation Bureau of Standards as *Proceedings of the Fourth International Symposium on the Chemistry of Cement.*

Participation

Participation in the Symposium was by invitation extended chiefly to persons actively concerned with research and development in cements and concrete. The three previous symposia were held in London, 1918; Stockholm, 1938; and London, 1952. The Symposium accepted an invitation from repre-

sentatives of Tokyo, Japan, to hold the next meeting in that city at a date to be announced.

Skokie seminars

Each of the 3 days at the PCA Skokie laboratories began with a tour of one of the facilities followed by discussions and seminars. The topics of the discussions and seminars included road research and product development, manufacture of cement, destructive processes in concrete, structural research, chemistry of hydration, structure and properties of paste, analytical procedures, chemistry of clinker, fire research, and the long-time study of cement performance in concrete.

A side trip to the AASHO Test Road at Ottawa, Ill., Oct. 13, gave the overseas visitors another opportunity to see American progress in highway construction.

Michigan Chapter Discusses Pavements

At its February meeting, the Michigan Chapter of ACI devoted the technical portion of its program to a discussion of concrete pavements. Maurice Greenberg, Engineer of Tests, Consolidated Laboratory Bureau, City Engineers Office, City of Detroit, spoke on "History of Paving Concrete and Concrete Control in Detroit." He covered briefly the history of street and sidewalk construction in Detroit from the era immediately after the Detroit fire before the Civil War to the present. Discussing the various types of street surfacing used in the early days, he noted that in 1877 Detroit placed its first "concrete" for a street. The concrete was used as a base material over which sand was spread and cedar block paving was laid. The first real concrete paving was in 1899 and was a private paving project using granite aggregate. This area was then deeded to the city and is still being used as a street today and has not been resurfaced. In 1909, Wayne County paved with concrete a portion of Woodward Avenue, Detroit's "main stem," that is believed to be the earliest concrete pavement in the United States and perhaps the world. The City of Detroit also started paving with concrete at this time and a street paved in 1912 is still in service and has not been resurfaced.

Mr. Greenberg also discussed early construction methods and compared them with those used in 1960. Some of the early concrete paving, the mixing and placing of which was by hand, used concrete so stiff that it had to be tamped in place. These methods were compared with today's modern machinery. Quality control and testing used in the two eras were also compared.

RILEM International Symposium to be held in Prague in July

The 15th meeting of the Permanent Committee of RILEM (Reunion Internationale des Laboratoires d'Essais et de Recherches sur les Materiaux et les Constructions) will take place in Prague on July 31 at the Philosophical Faculty of the Charles University. Between August 1-3 a symposium on the Durability of Concrete will be held dealing with: (1) theoretical problems of durability of concrete; (2) mechanics of concrete-texture deterioration; (3) durability tests and control; (4) composition and manufacture of durable concretes; and (5) experiences with concrete structures.

French and English are the official languages of the symposium. A simultaneous interpretation of both languages, as well as Russian will be given.

Following the symposium two excursions connected with the inspection of some important buildings and constructions will be organized: (1) to the famous spas of western Bohemia, August 4-6; and (2) to Moravia and Slovakia, August 7-12.

The symposium is being prepared within the framework of RILEM by the Czechoslovak Academy of Sciences, Institute of Theoretical and Applied Mechanics, Prague.

Automation for cement blending processes

Mixing and blending, such as in the cement and fertilizer fields, has long been an "art" but may succumb to automation according to statements in a paper, "A Study of a Generalized Model for Digital Computer Control of Blending Processes," presented by G. E. Adams, General Electric Co., Phoenix, Ariz., to the American Institute of Electrical Engineers.

The recent development of digital control computers and of new types of sensors (such as the x-ray analyzers) has made possible the application of the techniques of automation to blending processes. Digital computer control of blending processes has many advantages according to Mr. Adams, particularly in new plant construction. However, he believes that even in existing plants, the improvement in control over the quality of the finished product may justify the additional sensor and control equipment required.

Ellington honored

Harold S. Ellington has been awarded Honorary Membership in the Detroit Chapter of the American Institute of Architects.

Mr. Ellington, president of Harley, Ellington, Cowin and Stirton, Inc., Detroit, and former president of the Engineering Society of Detroit, was cited for his "immense influence in our public and civic affairs" and efforts to advance the architectural and engineering professions. Mr. Ellington has been an ACI member since 1948.

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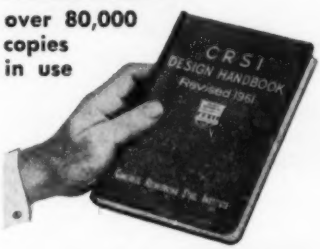

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CONCRETE PERFORMANCE REPORT:

Controlled performance concrete was needed for TWA Terminal roof shell

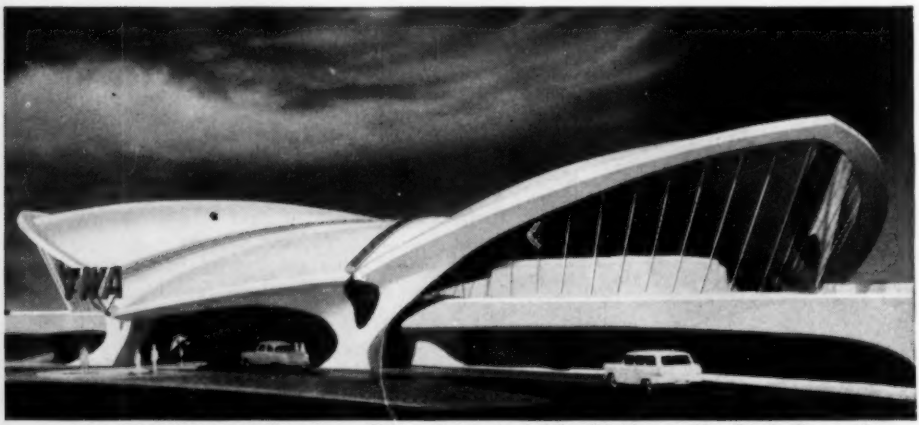
The 5500 ton roof of Eero Saarinen's distinctive TWA terminal at Idlewild Airport is an arch cantilever design—with four monolithic concrete shells, joined integrally at the centerplate, flowing out of four sculptured concrete buttresses.

In proportioning the 4000 psi conventional concrete for the buttresses and lightweight concrete for the shells, special consideration was given to these factors: (1) lowest possible unit water content and low cement content to minimize shrinkage (2) careful control of slump which had to be varied for different areas of the shell (3) precise control of setting time to assure a completely monolithic structure for each shell and to permit scheduled removal of counterforms (top forms) to allow finishing the surface to the final architectural contours. Also, in the buttresses and thick sections of the shell, a low rate of heat evolution was required to minimize thermal stresses.

Preliminary work showed that these objectives were best met with POZZOLITH concrete. The trap rock concrete for the buttresses contained 606 lbs. cement and POZZOLITH Retarder. The Norlite lightweight concrete for the shells contained 614 lbs. of cement, POZZOLITH Retarder and 23 gallons of water for the 5″ slump mix required under the counterforms. By simply regulating the amount of

TWA

POZZOLITH Retarder for each individual load of concrete, the set was retarded, as required, in the range of one to four hours. The two batching stations got their instructions by field telephone from the concreting superintendent on the roof.



EERO SAARINEN'S MODEL of new TWA unit terminal, Idlewild Airport, New York. Two-and-one-half year project scheduled for completion in late 1961. 50,000 sq. ft. lightweight concrete roof shell—supported by only four concrete buttresses—weighs 5500 tons, contains 3200 cu. yd. of POZZOLITH concrete and 500 tons of steel. Architect: Eero Saarinen & Associates • Supervising Resident Architect: Ralph Yeakel • Consulting Engineers: Ammann & Whitney—Boyd G. Anderson, Senior Partner in Charge • General Contractor: Grove, Shepherd, Wilson & Kruge, Inc.—Kenneth P. Morris, Field Project Manager • Testing: Haller Testing Laboratories, Inc.

NEW $15 MILLION SCULPTURE IN CONCRETE is as functional as it is breathtaking. Thickness of the lightweight concrete roof shell ranges from 8″ at the edges to 44″ at the centerplate. Each of the four huge shells is a continuous monolithic section.

TERMINAL

Concrete was placed in a round-the-clock operation averaging 27 hours per shell. Removal of counterforms proceeded on schedule. The concrete finished easily, with good texture and no plastic cracking. There were no shrinkage cracks in the entire shell. 28 day strength tests averaged 5000 psi.

Master Builders fieldmen worked closely with the resident architect, engineer, contractor and ready-mix producer at the job site to achieve the common goal of maximum quality at lowest cost through controlled concrete performance.

Write for your free copy of the detailed "TWA Terminal Concrete Performance Report".

The Master Builders Company
Division of American-Marietta Company
Cleveland 18, Ohio
World-wide manufacturing and service facilities

Our 50th Year

MASTER BUILDERS.
POZZOLITH* P-1068

*POZZOLITH *is a registered trademark of The Master Builders Co. for its ingredient for concrete which provides maximum water reduction, controls rate of hardening and increases durability.*

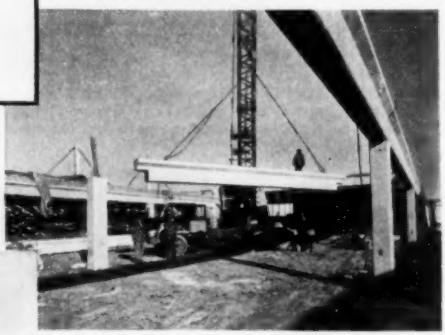
FOUR CONCRETE BUTTRESSES support the massive shell. There are no internal columns. Buttress section is shown below after form stripping. Buttress surfaces will later be bush-hammered to the texture selected by Architect Eero Saarinen.

Prestressed members, textured tilt-up walls

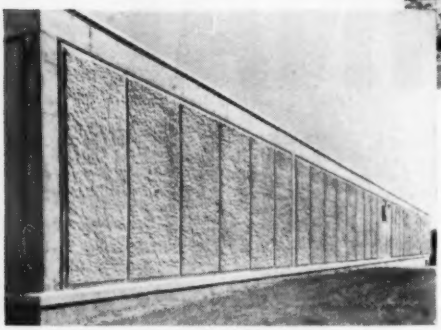
FOR NEW ALL-CONCRETE CANDY FACTORY

Owner:
Pearson Candy Company, St. Paul, Minn.
Architect:
Thorshov & Cerny, Inc., Minneapolis, Minn.
Contractor:
C. Q. Field Co., Minneapolis, Minn.

Prestressed Units:
Prestressed Concrete, Inc., Roseville, Minn.
Ready Mix Concrete For Tilt-up Slabs:
Guaranteed Concrete Company, St. Paul, Minn.
Light Weight Concrete Decking:
Western Mineral Products Co., Minneapolis, Minn.



(*Above*) The 160′ x 390′ building is framed on 30′ x 30′ and 39′ x 30′ column centers. Beams are prestressed, columns precast. The floor area at the two story kitchen supports live load of 225 lbs. per sq. ft. without reduction in column spacing.

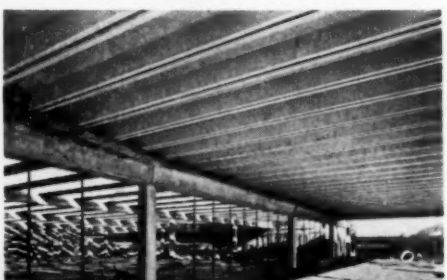


(*Left*) Most striking outward feature of the new Pearson factory is the tilt-up concrete wall. The interesting texture was achieved by casting the slabs on beds of gravel which had first been covered with plastic film. A total of 47 slabs 12′ high and of lengths from 5′ to 39′ was required.

• Prestressed beams, channels, and precast columns provide the structural strength and large open floor areas needed for the Pearson Candy Company operation at St. Paul, Minn. Lightweight concrete roof decking and interesting tilt-up walls provide insulation and architectural effect, and complete the fireproof and low maintenance qualities of the building from roof to foundation.

Lehigh Cements were used throughout. Lehigh Portland for tilt-up walls. Lehigh Early Strength for precast and prestressed members, and for the roof decking. Lehigh Portland Cement Company, Allentown, Pa.

LEHIGH CEMENTS



504 prestressed channel slabs, each spanning 30′, were used in the roof. Over these, lightweight concrete decking was used to insulate and reduce dead load. The simplicity of design seen in this picture indicates the ease with which an efficient building can be erected, using modern concrete construction methods.

Positions and Projects

World-wide visitors view ACI headquarters

During the past year architects, engineers, and student groups from every continent in the world have inspected the Institute's headquarters in Detroit, offering further evidence of ACI's ocean-spanning prominence.

Two of the most recent European visitors were Lyubomir Fournajieff, vice-president of Committee for Architecture and Town Planning, and Marin Jermanoff, head of Department of Committee for Building, Sofia, Bulgaria.

The visitors register shows recent guests from California, Illinois, Michigan, and Ohio. A group of 33 students from the University of Cincinnati included the headquarters building in their recent tour of unusual concrete structures in this part of the country.

HRB conducts 40th annual meeting

Nearly 3000 scientists, engineers, and public officials concerned with highway transportation attended the 40th annual meeting of the Highway Research Board, January 9-13 at the Sheraton-Park Hotel, Washington, D.C.

Delegates heard over 200 papers reporting results of recent research in highway financing, engineering, construction, planning, traffic control, and administration.

January 9 was devoted to technical committee meetings with the opening general session on Tuesday morning, January 10. The broad theme of this meeting was "The Look Ahead."

Special ceremonies were held honoring the Transportation Corps, Department of the Army, which for 2 years has cooperated with the Board and with the American Association of State Highway Officials in the AASHO Test Road project near Ottawa, Ill.

Detroit CIB presents '60 Concrete Achievement Award

The 1960 Concrete Achievement Award sponsored by the Huron Portland Cement Co. on behalf of the Concrete Improvement Board of Metropolitan Detroit, has been awarded the Grosse Pointe Farms, Mich., architectural firm of Meathe, Kessler & Associates. This comparatively young firm in the Detroit area has gained a rather enviable reputation in the past few years for their interesting and enterprising use of concrete in some very beautiful buildings for which they have received recognition in national architectural magazines.

The award, in the form of an engraved plaque, was presented this firm in recognition of their fine utilization of concrete in structures.

Six Alberta firms announce amalgamation

Six firms have amalgamated to form the new company, Consolidated Concrete Limited with headquarters in Calgary, Alberta, Canada. The companies involved in the merger were Consolidated Concrete Industries Ltd., Jefferies Concrete Products (Calgary) Ltd., Jefferies Concrete Products (Lethbridge) Ltd., True-Mix Concrete Ltd., True-Mix Aggregates Ltd., and Peerless Rock Products Limited.

Two ACI members, R. M. Jefferies and L. A. Thorssen, are members of the newly appointed board of directors, comprised of representatives from each of the merging firms. Other company directors are A. S. Jefferies, E. J. Lutz, and R. F. Jennings.

Another ACI member, B. A. Monkman has been named executive vice-president and along with L. E. Smith, vice-president, will be responsible for the active management of the new company.

Detroit CIB elects officers

The Concrete Improvement Board of Metropolitan Detroit has elected E. T. MacCracken, Barton-Malow Co., chairman for 1961. John A. Tucker, Ready Mix Concrete, Inc., was re-elected secretary; Bernard E. Ross, Wayne County Road Commission, treasurer.

Directors elected for 3-year terms include James E. Hampton, American Aggregates Corp., Bernard E. Ross, Wayne County Road Commission, and Robert E. Wilde, American Concrete Institute.

On Thursday, January 26, the CIB meeting was held at the Engineering Society of Detroit, Rackham Building, and about 80 members and guests were present. In addition to the usual business of the meeting, the annual Huron Achievement Award was presented to the Grosse Pointe Farms architectural firm of Meathe, Kessler & Associates.

Admixtures for Concrete

• *By ACI Committee 212*

Concrete admixtures are classified into 11 groups: accelerators, retarders, air-entraining agents, gas-forming agents, cementitious materials, pozzolans, alkali-aggregate expansion inhibitors, damp-proofing and permeability-reducing agents, workability agents, grouting agents, and miscellaneous.

This 34-page report discusses each group and the important effects to be expected in using materials of each group.

Order from Publications Department, American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich.

Price 75¢

Kenneth D. Cummins, ACI technical director, presented a talk on Formwork of Structures based primarily on the recent report of ACI Committee 622, Formwork for Concrete. He augmented his talk with slides depicting failures due to improper construction of formwork and a number of pictures of interesting use of formwork in unique concrete structures including the Lambert Airport Terminal at St. Louis, Mt. Clemens savings and loan association building, Mt. Clemens, Mich., and St. Clemens Church in Centerline, Mich.

Following Mr. Cummins' talk, an interesting discussion was participated in by O. G. (Bill) Sharrar of O. W. Burke Co. and Paul S. Barton, formwork consultant, both members of ACI Committee 622.

O'Leary joins Dewey and Almy

Thomas F. O'Leary of Wellesley Hills, Mass., has been appointed as sales engineer for construction chemicals with the Dewey and Almy Chemical Division, W. R. Grace and Co., Cambridge, Mass.

AEC participates in radioisotopes conference

The United States Atomic Energy Commission cooperated with the Chamber of Commerce of Greater Pittsburgh and the Pittsburgh chapter of the American Nuclear Society in presenting a Radioisotopes for Industry Conference, February 22 and 23 at the Penn-Sheraton Hotel, Pittsburgh.

The conference provided information on important industrial applications of radioisotopes. Industry and Atomic Energy Commission speakers reviewed the management, economic, and safety considerations in the use of radioisotopes. Speakers discussed improvements in production efficiency obtained by radioisotopes in industrial tracing, gaging, and radiography.

Verbeck lecturing at U of I materials seminar

George J. Verbeck, manager, Applied Research Section, Portland Cement Association, Chicago, is presenting a series of eight lectures at a seminar on the Physical Chemistry of the Behavior of Materials offered by the Department of Theoretical and Applied Mechanics, University of Illinois, Urbana.

Mr. Verbeck is a graduate of both Lawrence College, Appleton, Wis., and the University of Chicago in physical chemistry. His researches concerning numerous aspects of cement and concrete technology have been published in the ACI JOURNAL and other technical publications.

The first lecture was given on February 16 with subsequent lectures planned for 2:00 p.m. on approximately every other Monday running through May 25.

Stepanek elected to Alpha board

Harold F. Stepanek, secretary-treasurer of Alpha Portland Cement Co., Easton, Pa., has been elected to the company's board of directors. He fills a vacancy left last August by the death of former board chairman and president, John F. Magee.

Lock Joint acquires Electro Chemical

Lock Joint Pipe Co., East Orange, N.J., has acquired Electro Chemical Engineering & Manufacturing Co., Emmaus, Pa., and its associated companies, in an exchange of stock.

Electro Chemical specializes in chemical resistant cements, linings, and coatings. According to Allan M. Hirsh, Jr., president of Lock Joint, the acquisition is an important step in Lock Joint's entry into the field of corrosion resistant construction materials. Electro Chemical will operate as a wholly-owned subsidiary of Lock Joint.

Huron names Rightenburg to key sales post

Don Rightenburg has been appointed Detroit division sales manager for Huron Portland Cement Co., Detroit.

PCA appoints Hansen to Paving Bureau

Appointment of Harold W. Hansen as senior planning engineer of the Paving Bureau has been announced by the Portland Cement Association, Chicago. He joined PCA on November 1, 1960.

In a career covering more than 20 years as a civil engineer, Mr. Hansen has held many responsible positions with both governmental and private organizations. He is a graduate of the University of Minnesota, Minneapolis.

Measuring, Mixing & Placing Concrete

RECOMMENDED PRACTICE FOR MEASURING, MIXING AND PLACING CONCRETE (ACI 614-59). This ACI Standard outlines good practices for measuring and mixing the ingredients for concrete and for placing the finished product. Practices designed to achieve maximum uniformity, homogeneity, and quality of concrete in place are given special consideration. Recommendations cover handling of aggregates, batching procedures, mixing requirements, handling of mixed concrete, methods of placing concrete in forms, and consolidation. 32 pp. $1.00 per copy, 50¢ to ACI members.

AMERICAN CONCRETE INSTITUTE PUBLICATIONS

LOOKING AHEAD

Apr. 10-14, 1961—Phoenix Convention, American Society of Civil Engineers, Hotel Westward Ho, Phoenix, Ariz.

May 11-13, 1961—Annual Convention, National Lime Association, Grand Hotel, Point Clear, Ala.

May 15-16, 1961 — 41st Annual Meeting, Society of American Military Engineers, Washington, D.C.

May 16-18, 1961—1961 Spring Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

June 25-30, 1961—64th Annual Meeting, American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

June 26-July 1, 1961 — 7th International Congress on Large Dams, Rome, Italy

July 3-6, 1961—RILEM Symposium on Durability of Concrete, Prague, Czechoslovakia

July 30-Aug. 13, 1961 — RILEM International Symposium on the Durability of Concrete, Prague, Czechoslovakia

Aug. 30-Sept. 2, 1961 — IASS-RILEM Shell Colloquim, Delft, the Netherlands

Sept. 4-6, 1961 — IASS Shell Colloquim, Brussels, Belgium

Oct. 16-20, 1961 — Annual Convention, American Society of Civil Engineers, Hotel Statler, New York

Oct. 15-19, 1961—7th Annual Meeting, Prestressed Concrete Institute, Brown Palace and Cosmopolitan Hotels, Denver, Colo.

Addenda

The February story "Double-Shell Roof without Walls" omitted the name of Nicolas Esquillan of Boussiron Enterprises, Paris. Mr. Esquillan was responsible both for the design and the construction of the Exhibition Palace of the Centre National des Industries et des Techniques described in that story.

Errata

The following corrections should be made in "Effects of Aggregate Size on Properties of Concrete," by Stanton Walker and Delmar L. Bloem which appeared in the March 1961 JOURNAL.

p. 1234 — In the seventh line of the third paragraph change "water-cement ratio" to "cement factor."

p. 1234 — In the 11th line of the third paragraph delete "water-cement ratio" and insert "cement factor (450 lb per cu yd)."

p. 1234—Fig. N referred to in the eleventh line of the third paragraph was inadvertently omitted. Fig. N appears below.

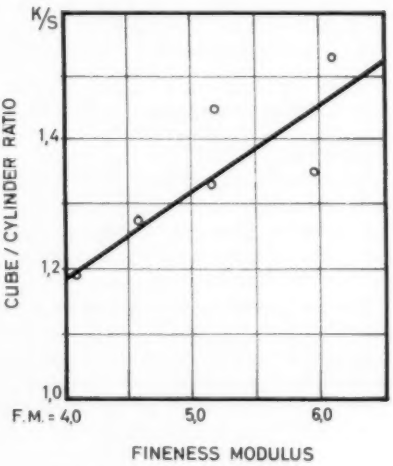


Fig. N—Gyenco test results

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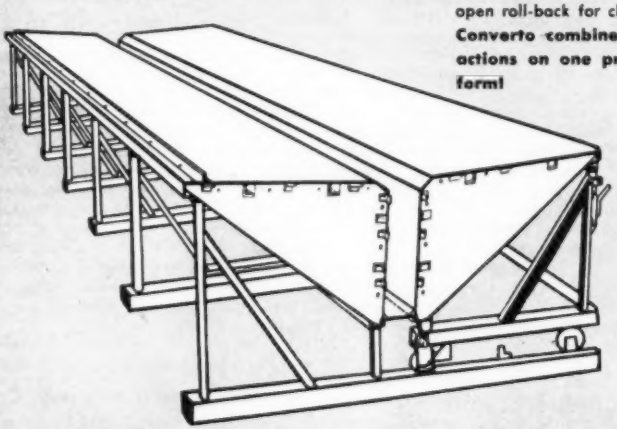

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Rate of Hardening of Mortars Sieved from Concrete Mixtures by Proctor Penetration Resistance Needles—ASTM C-403

The ASTM penetration resistance test of concrete is a procedure used to measure the rate of hardening of mortars sieved from concrete mixtures. The test is used to determine the effects of variables such as temperature, cement, mix proportions, additions, and admixtures on the hardening characteristics of concrete.
Fig. 1 (left) — The watertight and nonabsorptive test molds has a minimum depth of 6 in. and provides enough mortar surface for ten undisturbed readings of penetration resistance. **Fig. 2** (center) — In the preparation of the test specimen, the concrete is sieved over a No. 4 (4760 micron) sieve to provide enough mortar to fill the test mold to a depth of 6 in. The mortar is compacted by vibration or rodding in the test mold. **Fig. 3** (right) — The ASTM test involves forcing a steel needle into the concrete at regular time intervals and at a constant rate. The penetration force is measured on the penetration apparatus which is graduated from 10 to 130 lb. The force is converted to pounds of resistance per square inch. Needles with six bearing areas from 1/40 to 1 sq in. end area are provided. The larger needles are used first, with smaller needles used as the concrete stiffens. The calibrated steel needle of the penetrometer is uniformly forced into the concrete to a depth of 1 in. in approximately 10 sec. The 1 in. depth is indicated by a scribe mark on the needle. "Initial set" has been reached when the resistance to the needle penetration is 500 psi. "Final set" is defined as the point in the hardening process when the Proctor needle penetration reaches 4000 psi resistance. At this point, average concrete will have reached about 50 psi compressive strength. Photos courtesy Soiltest, Inc.

UA to offer graduate work in materials engineering

The Department of Mining and Metallurgical Engineering of the University of Arizona's College of Mines will offer the Master of Science degree in materials engineering beginning in September.

Materials engineering deals with the compatibility of the different kinds of materials used in the development of devices that utilize nuclear energy. Metals, ceramics, plastics, and adhesives, as well as wood and concrete are examples of materials involved in the work.

Students should have a bachelor's degree in engineering to be qualified for entering graduate work in this field. The Department of Chemical Engineering of the College of Mines, as well as other UA departments, will participate in the new graduate program.

Hinds appointed personnel director of Master Builders

William Hinds is the newly appointed director of personnel for The Master Builders Co., Cleveland.

Mr. Hinds has recently retired from active military service in the U. S. Air Force in the rank of Lt. Colonel. He returns to Master Builders having previously served as a sales representative for the firm from 1938-40.

Freseman joins ACPA staff

Howard P. Peckworth, managing director of the American Concrete Pipe Association, Chicago, has announced the appointment of Ray C. Freseman as assistant to the managing director.

Mr. Freseman is a graduate of the University of Minnesota, Institute of Technology, Minneapolis. For the past year he has been engaged as a design engineer specializing in foundation design for the Meissner Engineers, Inc., Chicago. For 2 years previous to this he was with Ingersoll Rand Co. as an application engineer in their Minneapolis branch.

Miller assumes new post at Austin Company

Loren C. Miller has been appointed assistant manager of the Chicago District of The Austin Company, international engineering and construction firm, Cleveland.

Mr. Miller, who has been works manager of Austin's Steel Fabricating Division in Cleveland, joined the Austin organization in 1949 immediately after graduating from Case Institute of Technology, Cleveland, with a bachelor of science degree in civil engineering.

CCI promotes two field engineers

Calcium Chloride Institute announces the promotion of two Institute field engineers, H. Bobbitt Aikin and Harry A. Smith, to the newly created positions of senior regional engineers. The position carries with it the duties of supervising field engineering, research, and other engineering and technical activities in assigned regions.

Mr. Aikin, from the Institute Washington, D.C., headquarters, has responsibility for the eastern-southern region. Mr. Smith, from Racine, Wis., will be in charge of the central-midwest region. The combined services of these two men amount to 13 years experience in calcium chloride usage, with specific emphasis on the concrete and highway industry.

NCMA appoints new autoclave engineer

Thomas B. Redmond, Jr. has been appointed to the newly created post of autoclave engineer for National Concrete Masonry Association. He will devote full-time attention to field studies, development research, and consultation on matters pertaining to high-pressure steam curing of concrete masonry units. He will headquarter at the NCMA office in Washington, D.C.

Mr. Redmond is a 1957 graduate of the University of Florida, Gainesville. Prior to joining the NCMA engineering staff he served as an engineering assistant to the director of research, Hydraulics Laboratory, Newport News Shipbuilding and Dry Dock Co., Newport News, Va.

Hubbard retires from NSA staff

Fred Hubbard, associated with National Slag Association, Washington, D.C., for the past 35 years, has retired as director of research and chairman of the NSA technical committee, and as consulting engineer for the Standard Slag Co., Youngstown, Ohio.

In recognition of his outstanding service to the slag industry, the Slag Association, at its 43rd annual meeting presented Mr. Hubbard with this association's first Award of Merit.

During the early part of his career he was employed by the Carnegie Steel Co. as field engineer and superintendent of construction. Here he had his first introduction to the use of blast furnace slag, primarily as an aggregate for portland cement concrete used as heavy steel mill foundations.

In 1925 he joined the staff of the Standard Slag Co. and since then has devoted his entire time to the develop-

ment and promotion of blast furnace slag as a commercial aggregate. During the past 35 years he has served NSA in many capacities; as a member of its board of directors, as director of research, and as chairman of its technical committee. He was the official delegate of the American Slag Industry at the International Conference on the Utilization of Blast Furnace Slag, held in Brussels, Belgium, October, 1954.

A recognized authority on the utilization of slag, much of Mr. Hubbard's research has been published in the ACI JOURNAL and other technical publications. He is currently a member of ACI Committee 115, Research, and Committee 613, Recommended Practice for Proportioning Concrete Mixes.

Illinois Highway and Traffic Engineering Conferences held

The 47th annual Illinois Highway Engineering Conference was held at the University of Illinois, Urbana, February 28-March 2, and the 13th annual Illinois Traffic Engineering Conference was held at the University on March 2-3.

Ellis Danner, professor of civil engineering, University of Illinois, was Highway Conference director. Some of the major topics discussed were quality control of concrete, economic forecasts, and relationships between urban planning and highway planning.

John Baerwald, professor of civil engineering, U of I, was Traffic Conference director. Major topics in this conference covered community planning and traffic and topics of general interest to traffic engineers.

Huron promotes Miller

The promotion of Blaine E. Miller to the position of assistant general sales manager, Commercial, Huron Portland Cement Co., Detroit, has been announced by Clarence L. Laude, vice-president in charge of sales.

Since 1957 Mr. Miller has been Michigan division manager of the company and has had charge of outstate Michigan and Indiana sales.

Lenawee Construction changes firm name

Lenawee Construction Co., general contractors, Adrian, Mich., has been renamed Goodrich-Hazen, Inc. There will be no personnel changes.

Zeevaert collaborates on embassy design

Leonardo Zeevaert, Mexico City architect, and Carolos Contreras have collaborated with the firm of Southwestern Architects and Engineers, Austin, Tex., on the design of the new U.S. embassy now under construction in Mexico City.

The $4.9 million building will have reinforced concrete walls covered with marble and aluminum. The structure, 221 x 150 ft with an interior patio of 90 sq ft, will have six floors and two penthouses, making it one of the largest U.S. embassies.

Kulka named to head Lin division office

Felix Kulka has been named to head a new San Francisco division office of T. Y. Lin & Associates, consulting structural engineers specializing in prestressed concrete.

Mr. Kulka, an associate of the firm since 1955, has been active in the design of prestressed concrete multistoried structures, particularly in the use of lift slabs, and in the design of thin shells. During his tenure with the organization he has been responsible for design work on such major projects as the prestressed thin shell roof of the Hipodromo Nacional, Caracas, the Norton Building, Seattle, the IBM Research Center, Westchester County, N.Y., and developments in Kuwait and Baghdad.

At present he is representing the firm as consultant to Anshen and Allen, San Francisco architects, on the projected University of California parking structure.

'61 Solite student awards announced

Three senior students of architecture at the Virginia Polytechnic Institute, Blacksburg, have won cash prizes totaling $750 in the 1961 Solite Awards Competition.

Lyles M. Harding, Jr., Norfolk, received first prize for his execution of the 1961 competition project. Herman J. Beckstoffer, Jr., Richmond, and Robin R. Dearing, Roanoke, placed second and third in the competition.

The contest sponsored annually by the Solite Corp., Richmond, is open only to senior students of architecture. It is primarily intended to encourage fresh and creative architectural design, particularly in the use of Solite lightweight structural concrete and masonry units. A similar competition is held at the University of Virginia, Charlottesville.

Master Builders transfers Kennedy to Fort Lauderdale

William T. Kennedy has been assigned as manager of The Master Builders Co.'s sales and field service facilities in Fort Lauderdale, Fla.

Mr. Kennedy has been with the company 6 years and for the past year has worked out of the division office at Miami. He is a graduate of Ohio State University, Columbus.

Bohn promoted to Dundee technical staff

Homer C. Bohn, one of the first sales representatives of the 2-year old Dundee, Mich., has been promoted to technical service engineer.

Mr. Bohn joined Dundee in 1959 several months before production actually began at the plant. He had previously served as a civil engineer with the Michigan State Highway Department, Lansing, and as sales engineer with American Prestressed Concrete, Inc., Centerline, Mich.

Francis A. McAdam

Francis A. McAdam, director of research and development of the Huron Portland Cement Co., Detroit, died recently at the age of 58.

Mr. McAdam joined Huron Portland Cement in 1954. Prior to that he was with Dewey and Almy division of Grace Chemical Co., Cambridge, Mass.

A structural engineer, Mr. McAdam headed the Nonmetallic Section, Building Materials Division of the War Production Board during World War II.

A native of Newport, R.I., he was a member of the American Concrete Institute, American Society for Testing Materials, and the American Mining Congress.

Maurice X. C. Weinberger

Maurice X. C. Weinberger, a senior partner in the consulting structural engineering firm of Weinberger, Frieman, Leichtman & Quinn, New York, died recently at the age of 80.

Mr. Weinberger received a degree in civil engineering from New York University. Following graduation he spent 6 years in the Phillippines working for the United States Government in the erection of bridges and the making of roads.

In 1923 he was a founder of Weissberger & Weisshoff predecessor of the present firm. Mr. Weinberger was active in the structural engineering work for the present Madison Square Garden and a number of theatres including the Ziegfield, Music Box, Capitol, and Loew's State. He also did the structural and apartment buildings and for engineering for many Manhattan office Loew's theatres in many parts of the world.

He was a member of the ACI, International Society of Bridge and Structural Engineers, and the American Welding Society.

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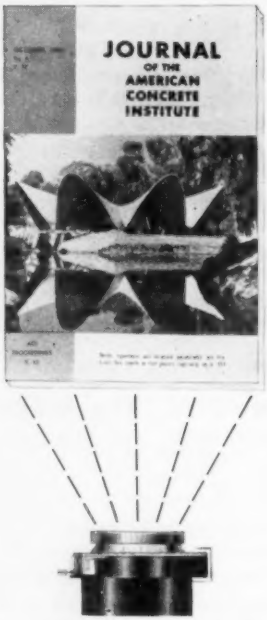

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Hanly retires

Hunter W. Hanly, rounding out some 55 years of civil and structural engineering experience, has announced his retirement from the firm of Hanly & Young, Cincinnati.

Having started his professional career in the construction field in 1906, when just out of college, he has been continuously engaged in concrete projects throughout the years. He has been an ACI member for the past 40 years. Mr. Hanly is now located in St. Paul, Minn.

Mr. Hanly's partner Truman P. Young will continue the firm under the name of Truman P. Young and Associates, civil and structural engineers.

NSGA and NRMCA conduct 15th annual short course

The National Sand and Gravel Association and the National Ready Mixed Concrete Association held the associations' 15th Annual Short Course on Concrete and Aggregates at the University of Maryland, College Park, the week of November 14-18.

The course was divided into three parts to permit some flexibility in selection of subject matter. Part 1 covered lectures and classroom work dealing with concrete technology. Speakers discussed the selection of concrete proportions; aggregate tests and specifications; factors affecting strength of concrete; lightweight concrete; and related subjects. Part 2 featured seminars for discussion of technical problems of the industries with emphasis on operations. Part 3 consisted of laboratory tests of aggregates and concrete. In addition to the NSGA and NRMCA staff, several outside speakers took part in the instruction.

One of the guest speakers at the short course, Richard C. Mielenz, presented a comprehensive discussion of "Admixtures and Other Aids for Concrete." Classifying admixtures into 12 types, Dr. Mielenz discussed each type

separately with regard to their functions in concrete, the general nature of their formulation, and the alterations in concrete characteristics which may be expected from their use. He thoroughly discussed water-reducers, subdivided into four classifications depending on their composition. He discussed factors involved in proper selection and use of these materials and the procedures necessary for their application to improve the competitive position of portland cement concrete and broaden its application.

Plotkin joins Truscon sales staff

Harold S. Plotkin, formerly associated with Sonneborn Chemical and Refining Corp., New York, has joined the sales staff of the Truscon Laboratories of Detroit and New York.

Dewey and Almy announce appointments

Richard T. Spada has been appointed to a newly-created position as special assistant to William M. Rand, Jr., Construction Chemicals sales manager of the Dewey and Almy Chemical Division, W. R. Grace & Co., Cambridge. Richard G. Allen will assume Mr. Spada's former position as midwest sales manager for the firm.

Crimm joins Admixtures as sales manager

Ralph G. Crimm, Jr. has joined Admixtures, Inc., Upper Darby, Pa., as sales manager. He had served for 4 years as sales and technical representative with the Dewey and Almy Chemical Division, W. R. Grace and Co., Cambridge. Dewey and Almy is the manufacturer of the construction chemicals distributed by Admixtures, Inc. and 21 other distributors in the United States and Canada.

Mr. Crimm is a member of ACI Committee 401, Specifications for Structural Concrete, a member of ASTM Committee C-9, Concrete and Concrete Aggregates, and a member of the American Association of Military Engineers.

Swayze retires from Lone Star

Myron A. Swayze recently retired from Lone Star Cement Corp., New York, after 26 years as director of research.

Graduating from Case Institute of Technology in 1912, Mr. Swayze joined the Lone Star Cement plant, Dallas, Tex., in 1913 as assistant chief chemist.

A Wason Medalist in 1942 for noteworthy research reported in the paper, "Early Concrete Volume Changes and Their Control," Mr. Swayze has authored numerous technical papers for publication in the ACI JOURNAL. Long active in ACI, he served as a director in 1944-45 and 1948-50, and on the Technical Activities Committee in 1951-1953.

Franke named a director of Penn-Dixie Cement

The election of William B. Franke, outgoing Secretary of the Navy, as a director of Penn-Dixie Cement Corp., New York, was recently announced by B. W. Bruckenmiller, president.

Mr. Franke has been active in governmental affairs since 1948. He served as assistant secretary of the Navy from 1954 to 1957 and under secretary from 1957 until his appointment as secretary in June, 1959.

PCI changes '61 convention date

The 7th annual meeting of the Prestressed Concrete Institute will be held October 15-19 at the Brown Palace and Cosmopolitan Hotel, Denver, Colo. The meeting was originally announced as scheduled for June 4-8.

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J. R. Sondhi	2

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The Board of Direction approved applications in the following categories: 76 Individual, 1 Corporation, 25 Junior, and 27 Student, making a total of 129 new member. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,375.

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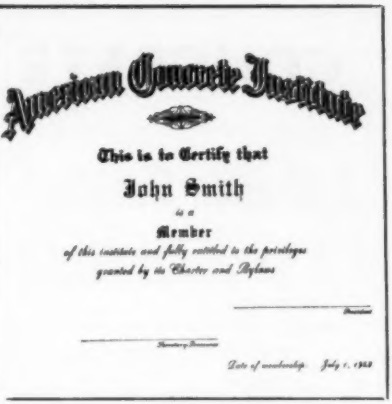
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Pipe lining firm expands services

Pipe Linings, Inc., specializing in rehabilitation of water pipe lines "in place" announces expansion into the Midwest, East, and Eastern Canada with headquarters established in New York. The firm is a 20-year old subsidiary of American Pipe and Construction Co. of Los Angeles.

In the last 5 years, the organization has cement mortar lined over 6,700,000 ft of pipe in diameters ranging from 4 to 180 in. using the Spunline and Tate processes, especially designed and developed for this work.—Pipe Linings Inc., P. O. Box 457, Wilmington, Calif.

Rubber rings interlock sewer sections

A system for permanently interlocking concrete sewer sections by means of rubber rings inserted in a groove between the units has been developed in Sweden. It has been evolved jointly by Trelleborgs Gummifabriks AB, who guarantee a life of 50 years to the rubber rings, and the makers of concrete working machinery, AB Cementvarumaskiner, Enstaberga. Engineered to strict standards as to hardness, breaking point, aging, and water absorption so as to outlast the lifetime of the sewers, the rubber rings are now delivered in all current dimensions from 150 mm to 1000 mm diameter.

Several other uses for rubber in the construction industry have been developed. They include the sealing of joints in houses built of prefabricated concrete elements, packing of various kinds of pipes and conduits, and leakage-preventing gadgets for use in tanks, valves and instruments containing certain fluid or gaseous matter.—Swedish Trelleborgs Gummifabrik, Stockholm, Sweden.

Epoxy resin bonding compound

Thiopoxy, a two-component, modified epoxy resin adhesive for bonding new to old, or old to old concrete, has been developed by Horn Research Laboratories.

Field and laboratory tests over the past few years show that this bonding adhesive is stronger than that which it bonds. Moderately damp conditions, or impact after aging, in no way causes failure due to brittleness or loss of adhesion according to Horn spokesman.

The developer recommends the adhesive for spalling or scaling concrete areas, broken slabs, or cracks. It is said to secure overlays of portland cement concrete surfaces, terrazzo, industrial floor toppings, sidewalks, and roadway resurfacing. Thiopoxy seals against water and salt intrusion, bonds traffic markers or extruded curbing to portland cement or bituminous concrete surfaces. It is

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a bonding agent on steel or concrete surfaces prior to using a gunite grout. Complete technical bulletin available.—Sun Chemical Corp., 750 Third Ave., New York 17, N. Y.

Square vibrating head

The new WYCO concrete vibrator features a square vibrating head. Manufacturer claims the square-head unit produces a strong five-

sided push in the concrete affording greater and faster compaction of stiff mix concrete according to supervised, accredited tests. The vibrator is available in ¾, 1, 1⅜, and 1¾ in square sizes.—Wyzenbeek & Staff, Inc., 223 N. California Ave., Chicago 12, Ill.

Extra-long-term equipment lease plan

Rock and cement companies can now lease equipment for terms as long as 6 to 12 years under a new plan announced recently by Nationwide Leasing Co., Chicago.

Companies, possessing a tangible net worth of $1 million are eligible for the extra-long-term leases for as little as $25,000 worth of equipment. There is no maximum.

The new plan features level quarterly payments for the term of the lease. No security deposit is required. Qualifying companies must show a 5-year record of profitable operations and equipment desired must have a depreciable life greater than the lease term.—Nationwide Leasing Co., Department LT, 11 South LaSalle St., Chicago 3, Ill.

Testing apparatus

Greater mechanical stability has been built into a new design of the Gillmore Apparatus for cement testing now being made by Soiltest, Inc.

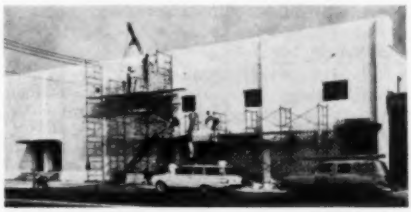
The apparatus determines the time of set of cement by determining at what time test specimens of the cement will bear the point of weighted needle. A ¼ lb needle determines the time of initial set while another needle weighing 1 lb determines the time of final set. In its redesign of the apparatus, the manufacturer has employed one-piece construction of the cross arms and needle supports to eliminate possible binding of the needles. It has also used a pin guide arrangement to prevent a rotation of the cross arms that could deflect the proper angle of the needles.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

Standard building products for playground use

A new use for standard concrete drainage units has been found by Lembo Concrete Corp., Huntington Station, N.Y. The company has introduced a line of pastel tinted playground equipment which has been converted from standard drainage forms. The new line contains a group of concrete boxes and rings in ladder like form which can be placed in backyards, community playgrounds, and parks. The three basic shapes are cube, cylinder, and cone.—Lembo Concrete Corp., Huntington Station, New York.

Precast concrete filigree panels

Achitectural excitement was created for the Casa Seville restaurant-nightclub in Franklin Square, Long Island, N. Y., by the use of a reinforced concrete filigree panel. Known as Dorian Sculp-Stone the panels



were used to build a three-dimensional facade of attention compelling design.

The panels for the Casa Seville job were fitted with dove-tailing notches and nipples. Each panel fitted into the panel beneath it. Specially designed steel brackets were bolted to the panels and the stucco wall behind it. The panel is 1 x 6 ft, 1¾ in. thick, with steel reinforcement throughout every member.

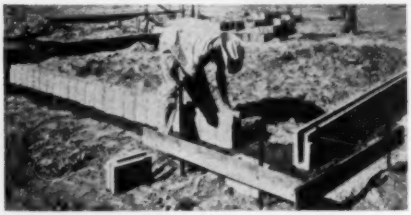
The basic panel is priced competitively, by the square foot, with the common filigree block. The panels have been used for such varied applications as walls and facades, fences, balcony railings, room dividers, lobby

ornamentation, carports, and background screening.

Complete pattern and specification information can be obtained from the manufacturer.—Dorian International, Roslyn, Long Island, N. Y.

Masonry form for slab-on-ground

A residential slab floor foundation procedure eliminating wood forming is being used in southern California. The procedure (patent applied for) has as its core two special foundation block invented by Tru-Bloc Concrete



Products Co. The units are called the "J" block and the "U" block.

The procedure is said to speed up slab completion by 1½ days. The procedure has been approved by the counties and major cities of southern California and is acceptable to FHA and VA in the San Diego area.

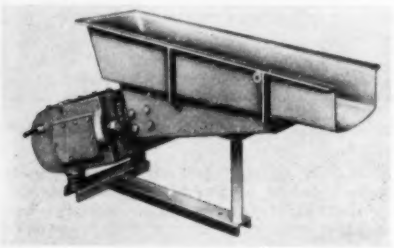
Basically, there is no departure in job layout from conventional lumber-formed foundations. Briefly the procedure is:

The trench is dug as usual, according to local code requirements, and the foundation block strung around the perimeter. Batter boards are placed and lines strung to required height. Then the footing is placed, as required, and the "J" blocks, 8 in. high on the inside, 12 in. high on the outside, are immediately set into the wet concrete of the footing, and brought to the line. The "U" block used for the garage area is placed in the same manner.

The area inside the foundation is filled and leveled to the 8 in. level of the "J" block, and then the slab is placed.—Tru-Bloc Concrete Products Co., 5960 Fairmount Extension, San Diego, Calif.

Heavy-duty vibratory feeder

Syntron has introduced a heavy-duty Vibratory Feeder, Model FH-22, with maximum capacity of 40 ton per hr to bridge the gap between the company's Model F-22 (25 ton per hr) and Model F-44 (50 ton per hr). Capacity is based on handling damp sand or coarser materials weighing 100 lb per cu ft. Capacity may be greater or less depending on the material. Stepless control of feed rate can be adjusted from maximum capacity down to 10 percent of maximum capacity.—Syntron Co., 1471 Lexington Ave., Homer City, Pa.



White Thiokol based sealant

E-BOND #1008, a chemically-vulcanizing sealant, based on Thiokol's LP-32, is said to be suitable for all types of masonry work and general application in the building industry.

The sealant is pure white and can be tinted to any shade of any color except red. It has tested to an elongation of more than 150 percent even after heat-aging equal to 15 years' exposure according to the developer. It is a two-part system with a simple mixing ratio. The product is also available under the company's Frozen Program in premixed, ready-to-use cartridges. ASTM specifications to which the product was tested are available on request.—International Epoxy Corp., 501 Northeast 33rd St., Fort Lauderdale, Fla.

Core length diameter indicator

A measuring indicator for quickly and accurately determining the length and diameter of drilled concrete cores to be used for testing is now available from Soiltest, Inc.

Concrete samples are core drilled from pavements, floors, walls and similar structures, and the samples used for checking the complete work with construction specifications as to thickness, quality, and strength. The new measurement indicator is designed to enable highway and construction engineers to check cores rapidly and with consistently accurate results. The indexing rotary table locks into 8 positions for measuring the core length up to 12 in. The portion of the indicator for measurement of diameter is detachable, thus making two independent measuring units for diameter and length.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

Drain and sewer tile testing machine

A 14,000 lb capacity precision tile testing machine has been introduced by Forney. Designated as Model PT-14, it is specifically designed for testing clay and concrete drain and sewer tile up to 24 in ID in lengths up to 48 in.

Equipped with two pumps, one for fast loading, and the other for applying the critical load slowly and with no pulsation, tests can be conducted in accordance with ASTM

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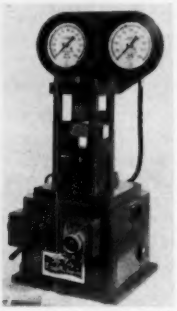
specifications, and with completely dependable results according to company spokesman.

The pressure gage is of precision design with a dial 8 in. in diameter calibrated 0 to 14,000 lb with 50 lb increments.—Forney's Inc., Tester Division, P. O. Box 310, New Castle, Pa.

Concrete tester

Transverse testing machines for determining the flexural strength of concrete specimens are now available from Steel City Testing Machines, Inc.



Designated Model TF-179, the bench-mounted testers have a capacity of 2000 lb. They apply a uniform load at a pre-set rate, which can be varied to suit different requirements. Flexural tests confirm to ASTM standards. The load is set at 500 lb per min. The machines are hydraulically operated, with the hydraulic system located within the base and powered by 1/6th hp motors operating on 110 v, 60 cycle alternating current. Simple controls are conveniently located on the front of the base for ease of operation.—Steel City Testing Machines, Inc., 8817 Lyndon Ave., Detroit 38, Mich.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

BE READY FOR READY-MIX CONCRETE—Four-page folder serves as guide in using ready-mixed concrete, highlighting various steps to insure easy placing and finishing, good workability and dependable results.—Huron Portland Cement Co., Technical Services Dept., Ford Building, Detroit 26, Mich.

CEMENT-TREATED SUBBASE FOR CONCRETE PAVEMENT—PCA 13-min film, prepared specifically for highway engineers and contractors, shows the various methods that are used to construct cement-treated subbases for concrete pavements. Emphasis is placed on finishing these subbases to accurate grade and crown. The advantages of cement-treated subbases are mentioned briefly. Cleared for use on television. $56 including can and shipping case.—Portland Cement Association, 33 West Grand Ave., Chicago 10, Ill.

JKS DIAMOND SAW BLADES—Brochure on diamond saw blades described as unique in the field for the amount and character of specific application data it presents. The brochure also introduces the JKS Pacemaker series of diamond saw blades, which includes segmented blades in five grades with diameters of 8 in. and larger, and continuous rim saw blades in two grades and in diameters of 3 in. and larger.—J. K. Smit & Sons, Inc., Murray Hill, N. J.

NATURAL FREQUENCY VIBRATING GRIZZLY FEEDER (Bulletin C900P)—Eight-page bulletin describes Carrier Amplitrol Grizzly Feeders, designed primarily for use in rock products and ore processing plants. Describes the units exclusive amplitrol drive that uses natural frequency to produce more work using less power; also outlines the feeder's fast-acting, remote control system. — Chain Belt Co., Public Relations—Publicity Dept., Milwaukee 1, Wis.

REX TRUCK-MIXER MODELS—Three bulletins describing each of the three new Rex truck-mixer models are now available: Bulletin 6177P on the Model 77, the economy model of the series; Bulletin 6142P on the Wate-Saver model, designed to permit greater legal payloads; and Bulletin 6155P on Model 155, designed for steady, high-production output under severe operating conditions.—Chain Belt Co., Sales Promotion Dept., Milwaukee 1, Wis.

CONCRETE MASONRY MANUAL — This manual presents in concise form the basic facts about expanded shale masonry. Chapters are devoted to concrete masonry construction, suggested specifications, block manufacture, characteristics, texture, patterns and bonds, strength and weight, thermal insulation, fire resistance, sound transmission loss, sound absorption, mortar, volume stability, coatings, basement walls, estimating quantities, and blast resistant construction.—Expanded Shale Clay and Slate Institute, National Press Building, Washington 4, D. C.

SURVEY OF MANUFACTURING ACTIVITY IN AUSTRALIA—Comprehensive, 27-page report on building and construction materials industry in Australia prepared by the Commonwealth's Department of Trade. Industries specifically covered in the study include: portland cement, structural steel, building bricks and blocks, roofing tiles, fibrous plaster sheets, and paint. In each category, charts and statistical indices pinpoint factors affecting the industry in terms of trends, production and demand, delivery, competition, stocks, costs and prices, employment, and investment. — New South Wales Government Office, Dept. P., 680 Fifth Ave., New York 19, N. Y.

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